VOLUME 85 NO. ST3
MARCH 1959
PART 1

JOURNAL of the

Structural Division

PROCEEDINGS OF THE

AMERICAN SOCIETY
OF CIVIL ENGINEERS

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Journal of the

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Proceedings of the American Society of Civil Engineers

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RIVET AND BOLT COUNCIL RESEARCH

E. J. Ruble,[1] M. ASCE

SYNOPSIS

The paper discusses the aims of the Council and its methods of planning and conducting research. The various organizations and universities sponsoring the Council are listed. The investigations that have recently been completed or those nearing completion are summarized and a complete bibliography of all published reports sponsored by the Council is included.

Organization

The Research Council on Riveted and Bolted Structural Joints was formed in 1947 by representatives of interested groups to plan and conduct extensive research leading to safer and more economical design and construction of metal structures. The principal advantage of such a council is that it brings together for discussion and planning a diversified group of design engineers, steel fabricators, rivet and bolt manufacturers, erection engineers, college professors, metallurgists, research engineers and many other scientists.

The Council depends entirely for membership upon organizations interested in the behavior of riveted and bolted structural joints. The Council conducts its research on a budget of about $60,000.00 per year and the following organizations are its principal financial sponsors:

1. American Institute of Steel Construction
2. American Iron and Steel Institute
3. Association of American Railroads
4. Bureau of Public Roads
5. Canadian Institute of Steel Construction

Note: Discussion open until August 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1969 is part of the copyrighted Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. ST 3, March, 1959.

1. Research Engr. Structures, Research Center, Assn. of Am. R. R., Chicago, Ill.

6. Industrial Fasteners Institute
7. The Engineering Foundation
8. Illinois Division of Highways
9. Washington State Council for Highway Research
10. Pennsylvania Department of Highways

In addition to the direct financial sponsorship, the various universities where the research is being conducted, such as the University of Illinois, Northwestern University, University of Washington and Lehigh University, contribute the time of the professors, testing equipment and other services, which are considered very valuable and sizeable contributions.

The Council meets once a year, usually for three interesting and informative days in February, and is usually attended by about fifty members and visitors. The Council meetings are interesting as all groups enter into an open and frank discussion of the results of the research.

Research Projects

The Council has several investigations underway at the present time and the papers contained herein by Professor Munse and Messrs. Petersen, Chesson and Bell of the University of Illinois, Professor Vasarhelyi of the University of Washington and Mr. Hansen from the Engineer Research and Development Laboratory at Fort Belvoir, cover part of its work on the study of bolted joints and high strength steels. The other two papers by Messrs. Ball, Higgins and Kinney result indirectly from Council work. In addition to these papers, the Council has completed the following investigations:

1. The Effect of Bearing Pressure on the Static Strength of Riveted Joints. This research was conducted at the University of Illinois and it was determined that the ratio of allowable unit tension stress to unit bearing stress of 1 to 1.50 could be increased to 1 to 2.25 without any decrease in the static strength of the joint.
2. The Effect of Rivet Pattern on the Static Strength of Structural Joints. This investigation on structural plate joints was conducted at the University of Illinois with an aim of developing a rivet pattern which would result in the greatest strength or efficiency of the riveted joint. No definite conclusion regarding the superiority of one particular rivet pattern was reached. However, it was concluded that no rivet pattern can be expected to develop a greater efficiency than 75 to 85 per cent and that the well-known equation of $s^2/4g$ is sufficiently accurate for all practical purposes.
3. The Strength of Rivets in Combined Shear and Tension. This research was conducted at the University of Illinois and resulted in definite data on the strength of single rivets when subjected to various combinations of concurrent tension and shear loading.
4. The Effect of Grip on the Fatigue Strength of Riveted and Bolted Joints. This research was conducted at Northwestern Technological Institute and clearly indicated that the increased clamping action resulting from the cooling of the longer rivets increased the fatigue strength.
5. The Effect of Rivet Pattern on the Fatigue Strength of Riveted Joints. This investigation was conducted at Northwestern Technological Institute

to determine if the fatigue strength of a riveted joint could be increased by a change in the rivet pattern.

At the present time the Council has several other investigations, in addition to those being presented, which are either nearing completion or underway. These investigations are:

1. Fatigue Tests on Cumulative Damage in Structural Joints. The Council has often questioned whether the data obtained from laboratory fatigue tests of structural joints are indicative of what may be expected in an actual structure. In the laboratory the maximum and minimum stresses are usually maintained constant during an entire test, while in an actual structure, the maximum stresses may vary considerably from cycle to cycle. For this reason, the Council authorized an investigation at the University of Illinois to secure data on riveted joints subjected to a sinusoidal variation of maximum cyclic stress with a constant minimum cyclic stress, this stress pattern being based on actual stresses in railroad bridges as determined by the research staff of the Association of American Railroads.
2. Effect of Bearing Pressure on the Fatigue Strength of Riveted Joints. The Council realized that while the ratio of allowable unit tension stress to unit bearing stress of 1 to 1.50 could be increased for static loading conditions, it should not be increased for joints subjected to repeated loading until more data were available on the subject. For this reason, the Council has authorized an extensive investigation at the University of Illinois on this subject.
3. The Effect of Pattern on the Static Strength of Truss-Type Structural Joints. This investigation, which is being conducted at the University of Illinois, is a continuation of the work done on structural plate joints. However, the joints now being studied are fabricated from plates and angles and are similar to those found in actual trusses. The load is applied to the members through gusset plates with either rivets or bolts connecting the plates to the members.
4. Static and Fatigue Strength of Bolted Structural Joints. The assembly of structural joints using high-strength bolts is a fairly new method to the structural engineering profession, and results directly from the large amount of research on fabricated joints which has been conducted by the Council within recent years. The Council has realized the importance of this subject and has sponsored a large number of investigations at various universities and while the investigations on this subject have not been completed, the Council has released many progress reports on the subject in addition to the published reports shown in the bibliography. A few of the interesting projects now underway on this study are:

    a. The Study of Nuts. Some trouble has been encountered with the stripping of nuts, especially where the strength of the bolt is considerably higher than the minimum requirement and the strength of the nut is close to the minimum requirement. A project has been started at the University of Illinois to make a detailed study of nuts having different hardness, including both heavy duty and regular nuts.

b. The Study of Washers. Northwestern Technological Institute is studying the basic principles of washer design to determine if the strength of bolted joints can be increased by a change in the shape of the washer.

c. The Study of Large Bolted Splices. The availability of the 5,000,000 pound testing machine at Lehigh University has made it possible to study large joints, typical of those that might be used in extremely large bridge chord splices. This investigation also includes a study to determine if the unit shear values can be increased for bolted joints.

In conclusion I wish to state that the work of the Council is continuing and this work will undoubtedly lead to further new concepts resulting in safer and more economical structures.

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STRENGTH OF RIVETS AND BOLTS IN TENSION[a]

W. H. Munse,[1] A. M. ASCE, K. S. Petersen,[2] J. M. ASCE
and E. Chesson, Jr.,[3] J. M. ASCE

SYNOPSIS

A study was made to determine the fastener behavior in a tension-type structural connection assembled with either ASTM A-325 high strength bolts or rivets of A-141 rivet material. In this study such factors as the flexibility of the connected members, the magnitude of the initial clamping force in the fasteners, the number of lines of fasteners, and the grip of the fasteners have been considered.

INTRODUCTION

Whether rivets and bolts should be permitted to carry loads in direct tension has been discussed by engineers(1) for many years and, on occasion, some have strongly objected to their use in this manner. Even today, a few of our design specifications prohibit the use of structural connections which are subjected to direct tensile loads. And yet, from time to time, experimental studies have been conducted which demonstrate the ability of rivets and bolts to carry such loads effectively.

In an earlier study reported by Leahey and Munse,(2) a number of tests were conducted to determine the behavior of rivets and high-strength bolts loaded in direct tension by means of a structural Tee-connection, such as may be found in some heavy wind bracing connections, crane hangars, etc. Leahey studied the behavior, under static and fatigue loads, of this Tee-type

Note: Discussion open until August 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1970 is part of the copyrighted Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. ST 3, March, 1959.

a. Presented at the February 1958 ASCE Convention in Chicago, Ill.

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connection when assembled with two lines of fasteners, i.e., one line on each side of the web of the Tee. His study included such variables as the initial tension in the bolts, the spacing of the bolts, and the use of inclined bearing surfaces under the heads of the bolts.

Although Leahey's tests answered a number of questions concerning the behavior of connections with high-strength bolts when loaded in tension, many other problems still required study. Attention has been given to some of these factors in the tests reported herein. The flexibility of the members, the grip of the fasteners, and the effectiveness of the fasteners in members fabricated from wide-flange sections and assembled with two or four lines of fasteners have been examined.

Laboratory Tests

Specimens

Whereas the previous specimens had been cut from a 24 I 100, the members for this current series of tests were fabricated from several wide flange sections. In the previous tests it was noted that the prying action at the toes of the relatively thin flanges apparently reduced the capacity of the bolted connections to a value below the total tensile strength of the fasteners (as determined by tensile tests of the individual bolts). In order to consider this matter further, flanges of several thicknesses and widths were included in the present program.

In actual tension connections (connections of members such as the Tee-sections of this study) the total thickness or grip may vary considerably even though the stiffness of the connected sections is not changed. Since this variation in grip might affect the prying action, because of the greater deformation and elongation possible in longer bolts, a pilot test was made using a 2 in. fill plate between the flanges of a specimen (24 I 100). The test data indicated an increase in ultimate strength of more than 14 per cent with the increased grip. In order to investigate this matter further, many of the new test specimens were fabricated with a fill plate.

Four major variables have been included in the present tests: the number of lines of fasteners, the flange width, the flange thickness, and the type of connector. A summary of the specimens and a listing of the test sections are included in Fig. 1. The twenty-eight tests provided one riveted and one bolted specimen with a fill plate and similar specimens without fill plates for all specimen types plus two additional specimens, CB9* and EB9, which were added for a further study of the effect of flange stiffness.

The Tee-sections for the test specimens were of ASTM A-7 structural steel and the fasteners consisted of bolts meeting ASTM designation A-325 or rivets from material meeting ASTM designation A-141.

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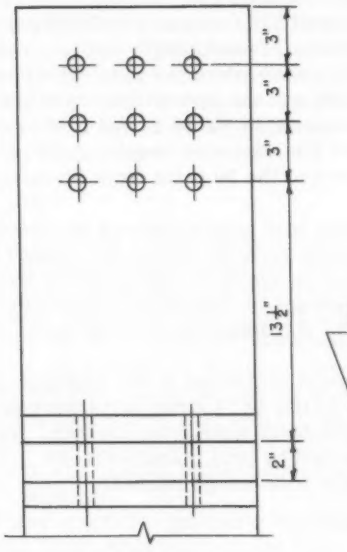
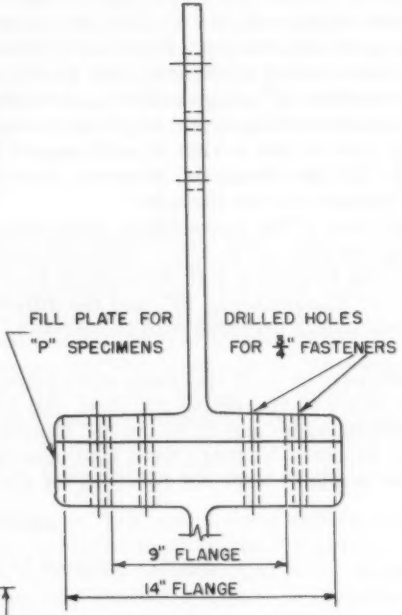
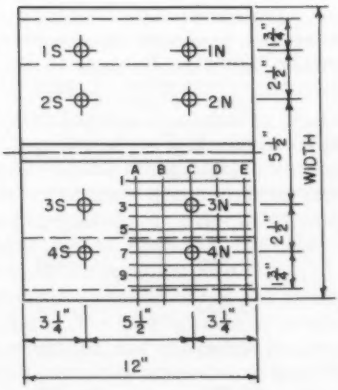
*Specimens were marked with the following code: The first letter designates the specimen type and size of the Tee-section (see Fig. 1); the second indicates whether the specimen was bolted (B) or riveted (R); if a P is included in the mark, the specimen was tested with a 2 in. fill plate between the flanges; V indicates variable prestress in the bolts; and the 9 refers to the 9 in. flange width used in the final two tests. Thus, riveted specimens AR, ARP, BR, . . . were identical in geometry to bolted specimens AB, ABP, BB, . . . respectively.







SPEC. TYPE	SECTION SIZE	FLANGE WIDTH	FLANGE THICK.	NO. OF TESTS
A	24WF130	9.00	0.90	4
B	24WF130	14.00	0.90	6
C9	30WF210	9.00	1.32	1
C	30WF210	14.00	1.32	4
D	30WF210	15.11	1.32	4
E9	36WF300	9.00	1.68	1
E	36WF300	14.00	1.68	4
F	36WF300	16.66	1.68	4

TOTAL TESTS 28

FIG. 1 DETAILS OF TEST SPECIMENS

Tests of full size bolts and of standard 0.505-in. round coupons cut from the bolts were conducted to obtain information on their properties. From the 0.505-in. coupons, it was found that the bolts of Manufacturer "N"* had strengths of 120,000 psi to 138,200 psi and averaged 126,600 psi. Coupons of bolts from Manufacturer "S" gave strengths of 126,000 psi to 146,300 psi with an average of 136,000 psi. Five full-size bolts of each length and manufacturer were also tested in tension. The average strength of the four bolts tested to failure with a 10° wedge under their heads and the strength of the other bolt tested to failure without the wedge are presented in Table 1. All of the bolts and nuts used in this series of tests passed the minimum requirements of ASTM A-325 Specifications; however, eleven of the 30 bolts from Manufacturer "S" stripped the nut threads.

In the tests of the connections, bolts from both manufacturers produced stripping failures of the nuts. These stripping failures probably resulted because of the relatively high strength of some of the bolts, particularly those supplied by Manufacturer "S", and the difference or variation in strength of the nut materials provided with those bolts. Hardness tests on the nuts

*The fasteners of each specimen were marked according to the orientation of the specimen in the testing machine. All of the "N" (north) bolts were from one manufacturer, and all of the "S" (south) bolts were from another. Nuts and flat hardened washers were also provided by both manufacturers. (Beveled washers were not required for the wide flange shapes).

TABLE 1

TENSILE STRENGTH OF FULL SIZE BOLTS

Bolt Length, in. (1)	Ultimate Tensile Strength, lb.			
	Manufacturer "N"**		Manufacturer "S"**	
	Wedged Avg* (2)	Not Wedged (3)	Wedged Avg* (4)	Not Wedged (5)
3	45,500	45,800	44,550	46,700
4	45,300+	45,500	42,550	42,400
4.75	47,350	47,100	48,520	48,300
5	44,150	43,800	45,890	43,800
6	46,820	46,000	45,700	44,600
7	44,720	44,800	49,220	49,600

* 10° wedge, average of 4 tests unless otherwise noted

\+ 10° wedge, average of 2 tests

** Bolts from Manufacturer "N" were the North bolts, while South bolts were supplied by Manufacturer "S"

furnished by Manufacturer "S" gave readings of 69 to 85 Rockwell B with an average value of 79 (approximately 143 Brinell). Similar hardness tests on the nuts from Manufacturer "N" produced a range of 90 to 98 Rockwell B with an average value of 95 (approximately 209 Brinell).

For the various rivets used, coupons of 0.252-in. diameter and 1-in. gage length were turned from driven but unfractured rivets which had been removed from the specimens after testing. The ultimate strength of these coupons are shown in Column 4 of Table 2. It was felt that these coupons would reflect better the strength of the rivets after driving than would coupons prepared from undriven rivets.

Assembly

The bolts for the connection tests were individually calibrated. In these calibrations the elongations and the torques required for the desired bolt loads were measured. As would be expected, the shorter bolts required somewhat less total elongation to attain 0.9 of the Elastic Proof Load (EPL) than did the longer bolts. There was some variation between individual bolts of the same length and manufacture, and between one manufacturer and the other; however, these variations were not great. The torque required to load the bolts to 0.9 EPL was found to be reasonably constant for all lengths, and equal to approximately 320 lb. ft.* The prescribed minimum tension was obtained at one-half to three-fourths of a turn of the nut from an initial "finger-tight" position.

Three electric resistance strain gages, 120° apart, were applied to the shanks of all eight bolts on the first specimen to be tested. In view of the uniformity of the average strains observed in each of the eight bolts of that test, it was decided to use strain gages on only two bolts in each of the subsequent tests with the exception of specimens designated EB9 and CB9; no strain gages were used on the bolts of the latter members. For a bolt with strain gages, the average strain found necessary in the calibrations to obtain the desired bolt tension was used to set the prestress in that bolt during assembly. For all of the other bolts (those without strain gages) the tension was set on the basis of the elongations obtained in the calibration tests.

All holes were match drilled and carefully aligned before the specimens were assembled. For the bolts with strain gages it was necessary to drill 15/16 in. holes; however, the other bolt holes and all the rivet holes were drilled 13/16 in. diameter.

During the assembly of the riveted members by a large structural steel fabricator, the inside rows of rivets were driven first, the fit-up bolts were then removed, and the outer rivets were driven. The bolted specimens were aligned carefully also, fit-up bolts were installed and then the test bolts were tightened in a predetermined sequence. Upon checking, the first bolts tightened in a specimen were sometimes found to be below the desired prestress and therefore had to be retightened; however, adjustments were made until all of the bolts had a minimum prestress of 0.9 the elastic proof load (except for specimens BBV and BBVP where only 0.1 EPL was used for the inside fasteners).

*It may be noted that this is the approximate equivalent torque for the minimum bolt tension recommended by the Research Council on Riveted and Bolted Structural Joints.

TABLE 2

RESULTS OF TESTS OF RIVETS AND RIVETED CONNECTIONS

Specimen	Equiv. Rivet Size	Ult. Load, Kips	Ultimate Strength of Rivets, psi	Pred. Load** Based on Rivet Strength, Kips	Efficiency, per cent	Eff. With Plate ÷ Eff. no Plate
(1)	(2)	(3)	(4)	(5)	(6)	(7)
			Specimens With Two Lines of Fasteners			
AR	3H+	141.3	83,600*	173.4	81.5	0.92
ARP	5H	104.4	67,350	139.7	74.7	
			Specimens With Four Lines of Fasteners			
BR	3C	164.4	89,600	371.7	44.2	1.08
BRP	5C	161.3	81,500	338.1	47.7	
CR	4C	210.0	76,500	317.3	66.2	1.13
CRP	6C	202.4	65,450	271.5	74.5	
DR	4H	211.5	73,950	306.7	69.0	1.07
DRP	6C	199.3	65,300	270.9	73.6	
ER	4.75C	254.6	82,700	343.0	74.2	1.04
ERP	7C	276.4	86,600	359.2	76.9	
FR	4.75H	212.0	67,050	278.1	76.2	0.99
FRP	7C	270.0	86,600	359.2	75.2	
					Average	1.04

+ H indicates rivets which were hot formed; C identifies cold formed rivets; all rivets were driven hot.

* Based on one coupon test; all other ultimate rivet strengths are based on two coupon tests.

** Based on coupon tests of driven rivets and actual hole diameters.

TABLE 2 (Continued)

Specimen	Type and Location of Failure	Distribution*** of Total Strain to Fasteners, per cent				Rivet Most Highly Strained
		North	South	East	West	
(1)	(8)	(9)	(10)	(11)	(12)	(13)
	Specimens With Two Lines of Fasteners					
AR	Shank 3S	53	47	41	59	2N
ARP	Shank 3S	49	51	59	41	3S
	Specimens With Four Lines of Fasteners					
BR	Head 2N	49	51	51	49	3N
BRP	Shank 2S	54	46	44	56	2N
CR	Shank 3N	48	52	49	51	2N
CRP	Shank 2S	50	50	59	41	3N
DR	Shank 2S	48	52	41	59	2S
DRP	Shank 2S	48	52	51	49	3S
ER	Shank 1S, 2S, 1N, 2N, 3N	51	49	53	47	3N
ERP	Shank 1S, 2S, 1N, 2N	53	47	45	55	2S
FR	Head 3S and 4N	50	50	48	52	2N
FRP	Head 2N	49	51	47	53	2S

***At maximum load for which records were obtained.

Test Procedures

Most of the specimens were tested in a universal testing machine of 300,000 lb. capacity. The load was applied in steps of 10,000 or 20,000 lb. and the readings or measurements mentioned below were taken after all load increments. Each specimen was loaded until the maximum load was reached and the first fastener failure occurred in one or more inside fasteners. The loading was then continued until all of the inside fasteners failed, or until one of the outside fasteners failed. As soon as an outside fastener failed the test was discontinued.

During the tests the following readings were taken: strains in selected bolts, elongation of all fasteners, and flange separations in one quadrant of all specimens. Small gage point holes (less than 1/16 in. deep) were drilled in both ends of the fasteners and in the flanges to receive the points of the extensometers that were used to measure bolt elongations and flange separations. The 3/8-in. by 5/8-in. grid of points for the flange separation measurements can be seen in Fig. 1.

Riveted Specimen ER is pictured in Fig. 2 upon completion of its test. In Fig. 3 may be seen the residual flange deformations remaining in several bolted specimens of identical flange width but varying thicknesses.

Test Results

Before discussing the test results it should be noted that in the analysis of the data, a number of factors must be considered. Because of the widely varying grips for specimens with and without fill plates and cut from different wide-flange sections, the ductility and strength of the fasteners must be kept in mind when comparisons are made of the results from the various tests. The differences in grip may affect the deformations which are produced by a given load; the longer bolts will probably deform more than the shorter bolts since two bolts of identical material but of two different lengths will not have the same elongation at a given load. Unequal loading of the bolts, because of eccentricities or warping in the specimen, also will have an effect on the behavior of the members. The webs of Specimen EB9, for example, before testing were found to be bowed almost 1 in. because of the "out of squareness" of the flanges.

The question of nut stripping failures must also be considered. Failure by thread stripping is not sudden and considerable load may be carried even after stripping has started. However, it is obvious that higher loads can be sustained by bolts for which the nuts strip, if the bolts are assembled with nuts strong enough to cause failure by bolt fracture. This point must be kept in mind when comparing the test results since many of the failures were initially by stripping. Finally, it should be remembered that the cross-sectional area of a driven 3/4 in. diameter rivet is about 0.518 sq. in. whereas the stress area of a 3/4 in. diameter bolt is 0.334 sq. in. Thus, the lower strength rivets may, in some cases, develop a load carrying capacity approaching that of comparable size high strength bolts.

The results of the tests are given in detail in Tables 2 and 3. Members having two lines of fasteners are listed first, followed by those having four lines of fasteners.

FIG. 2 SPECIMEN ER AFTER FAILURE

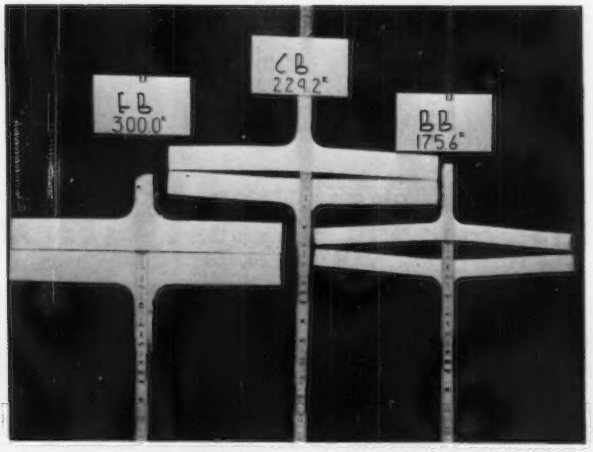


FIG. 3 PERMANENT DEFORMATIONS OF BOLTED MEMBERS AFTER TESTS

TABLE 3

RESULTS OF TESTS OF BOLTED CONNECTIONS

Spec.	Bolt Length, in.	Ult. Load, Kips	Ult. Load Bolt ÷ Ult. Load Rivet	Pred. Load Based on Bolt Strength Kips	Efficiency per cent	Eff. with Plate ÷ Eff. no Plate
(1)	(2)	(3)	(4)	(5)	(6)	(7)
			Specimens With Two Lines of Fasteners			
AB	3	134.0	.948	180.1	74.4	1.02
ABP	5	136.8	1.310	180.1	76.0	
CB9	4	175.0		175.7	99.6	
EB9	4.75	169.0		191.8	88.1	
			Specimens With Four Lines of Fasteners			
BB	3	175.6	1.068	360.2	48.8	1.03
BBP	5	181.6	1.126	360.2	50.4	
BBV	3	183.4		360.2	50.9	1.00
BBVP	5	184.3		360.2	51.1	
CB	4	229.2	1.091	351.4	65.2	1.02
CBP	6	245.7	1.214	370.1	66.4	
DB	4	219.6	1.038	351.4	62.5	1.11
DBP	6	256.0	1.284	370.1	69.2	
EB	4.75	300.0	1.178	383.5	78.2	1.03
EBP	7	302.7	1.095	375.8	80.5	
FB	4.75	274.1	1.293	383.5	71.5	1.07
FBP	7	286.7	1.062	375.8	76.3	
		Average	1.142		Average	1.04

TABLE 3 (Continued)

TABLE 3 (Continued)

Spec.	Type and Location of Failure	Distribution** of Total Strain to Fasteners, per cent				Bolt Most Highly Strained	Eff. Bolt / Eff. Rivet
		North	South	East	West		
(1)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
		Specimens With Two Lines of Fasteners					
AB	Nut 2S	56	44	25	75	2N	.913
ABP	Nut 3S	57	43	43	57	2N	1.018
CB9	Nut 2N, Bolt 2S	34	66	51	49	3S	-
EB9	Nut 3S	61	39	73	27	3N	-
		Specimens With Four Lines of Fasteners					
BB	Nut 3S	71	29	62	38	2N	1.107
BBP	Bolt 3N	55	45	56	44	3N	1.059
BBV	Bolts 2N and 3N	57	43	58	42	3N	-
BBVP	Bolts 2N, 3N and 2S	54	46	55	45	3N	-
CB	Nuts 2N and 3N	36	64	57	43	3S	.984
CBP	Bolt 2N	53	47	57	43	3N	.892
DB	Nuts 2N and 3N	31	69	61	39	3S	.906
DBP	Bolt 3N	53	47	54	46	3S	.939
EB	*	77	23	50	50	3N	1.056
EBP	Bolts 1N, 2N and 1S Nuts 2N and 2S	66	34	51	49	3N	1.047
FB	Nuts 2S and 3S	74	36	41	59	2N	.938
FBP	Nuts 2S and 3S	58	42	50	50	2N	1.016
						Average	0.990

* All fasteners failed simultaneously in the bolt except 2S and 3S which failed in the nut.

** At maximum load that records were obtainable.

Members With Two Lines of Fasteners

Two riveted and four bolted specimens with two lines of fasteners were tested. The type A specimens, AR, ARP, AB, and ABP, were fabricated from a 24 in. wide-flange 130 lb. section (the same rolled shape as that used for the type B specimens), specimen CB9 was fabricated from a 30 in. wide-flange 210 lb. section (the same section as the type C and D specimens) and specimen EB9 was fabricated from a 36 in. wide-flange 300 lb. section (the section used for the type E and F specimens). However, in each instance the flanges of the members had been cut to a width of 9 in.

In order to study the behavior of the bolts during the tests, the elongations of the bolts were determined with SR-4 strain gages or by means of a mechanical extensometer. From these measurements it could be seen that the load was not always distributed uniformly to the bolts, a condition that might affect the ultimate load carrying capacity of the members. However, it was noted that the most heavily strained bolts did not always fail, particularly in the cases where failure was by thread stripping.

The more sensitive SR-4 strain gage measurements indicated small changes in bolt tension or bolt strain at a load of 4000 to 5000 lb. per bolt. However, elongation measurements made with the mechanical extensometers did not reflect noticeable changes in bolt load until the loads on the bolts averaged as much as 10,000 to 20,000 lb. per bolt. Columns 9 through 13 of Tables 2 and 3 indicate the manner in which the load was distributed to the rivets and bolts during the latter stages of testing. However, when compared with the information in column 8 of the same tables, the location of the failures, it is found that in only several instances did the most highly strained bolt or rivet provide the point of failure. Thus, the unequal distribution of load to the fasteners is certainly not the only factor affecting the strength or efficiency of the connections.

Stripping of the nuts or shearing of the rivet heads causes a decrease in the elongation of the affected fasteners and, at the same time, provides a weakened location at which failure can initiate. When this type of shearing takes place in the fasteners, the applied load will be redistributed and produce increased strain in the other bolts or rivets. Such behavior also seems to be reflected in the data of Tables 2 and 3.

From the bolt elongation plots for Specimens AB, CB9, and EB9 in Fig. 4 (right hand edge of upper diagram) it will be noted that the bolts in CB9 and EB9 deformed less for a given load than did the bolts in AB. The stiffer flanges probably reduced the effect of prying action considerably, thereby allowing more of the load capacity of the bolts to be utilized in carrying the applied load. In addition, the bolt elongation in specimens CB9 and EB9 were further reduced at any given load because the bolts in these specimens had a yield strength about 10 per cent greater than the bolts in Specimen AB. Further confirmation of the reduced prying action with the stiffer flanges is obtained from the fact that specimens CB9 and EB9 developed almost the full capacity of the individual bolts, although unequal distribution of the load to the bolts and nut stripping failures no doubt reduced the efficiencies of these specimens somewhat.

Members With Four Lines of Fasteners

The B specimen may be considered as the basic member; then, all others vary from this basic type by thickness or width of flanges and, of course, by the fasteners used.

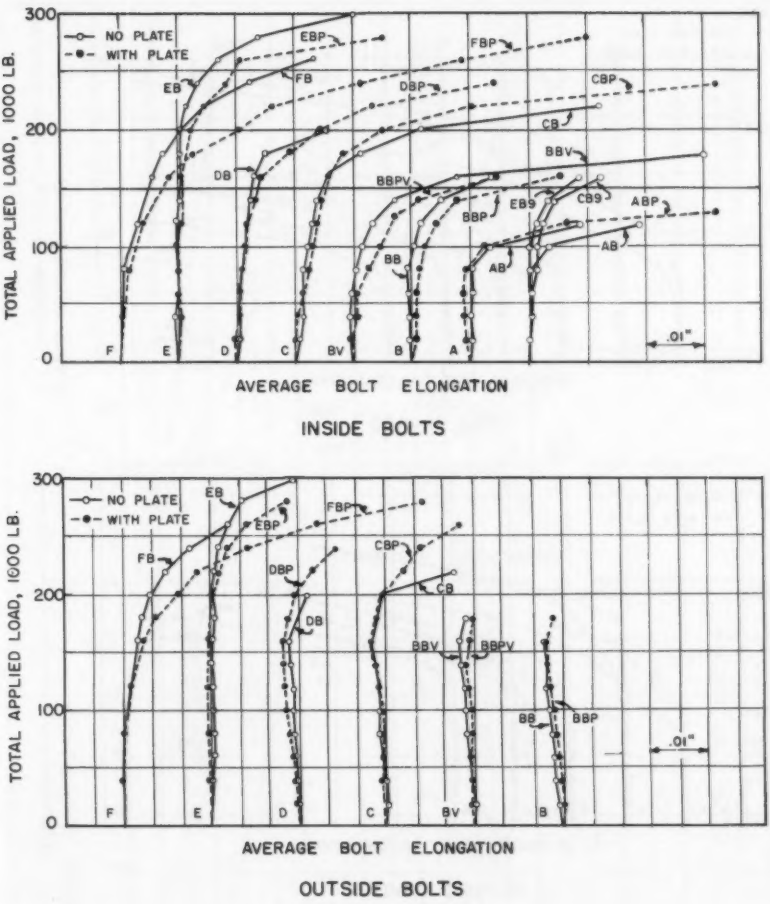


FIG. 4 CURVES OF LOAD VS. AVERAGE BOLT ELONGATION

The strain in the inner bolts of specimen BB began increasing at a total load of 10,000 lb. and continued to do so until failure occurred. (See Fig. 5). However, the strain in the outer bolts decreased slightly as the member was loaded to 160,000 lb., and then began to increase. Thus, in the early stages of loading the entire load is carried by the inner bolts.

An additional indication of the behavior of the members may be obtained from the flange separation measurements. It is interesting to note in Fig. 6 that point E7 (the outer bolt line) of specimen BB did not exhibit a flange separation until the load reached 160,000 lb., further indicating the unequal distribution of bolt load in the inner and outer bolt lines. The flange separation at

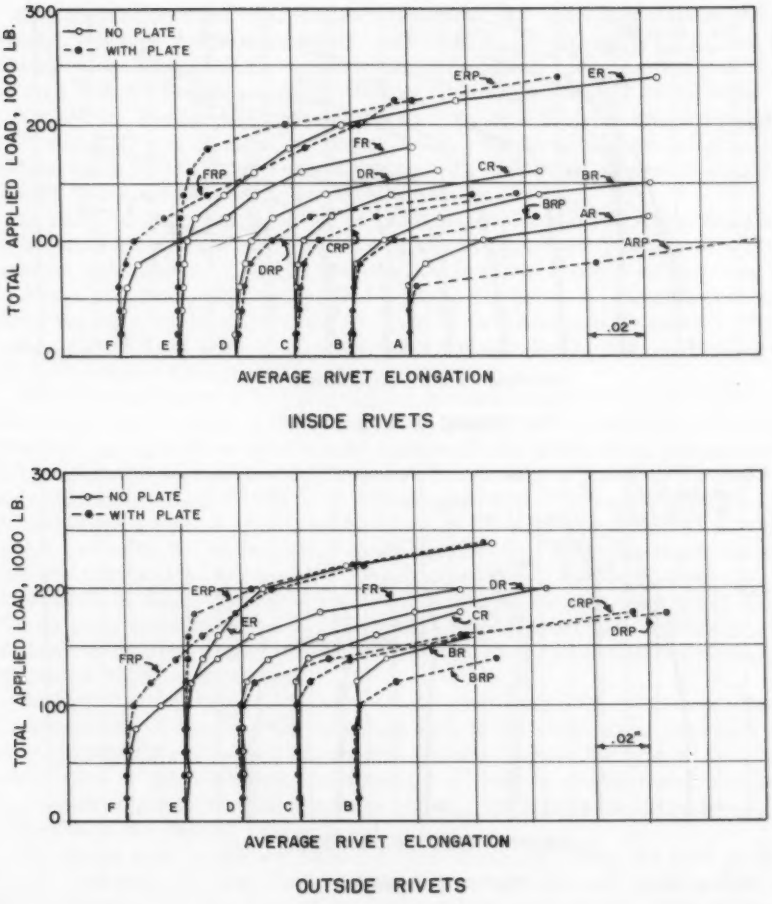


FIG. 5 CURVES OF LOAD VS. AVERAGE RIVET ELONGATION

the web was large, but the edges of the flanges remained in contact at most points throughout the entire test. Failure occurred in Nut 3S, one of the nuts on the inner bolt lines, at a load comparable to 48.8 per cent of the total strength of the eight fasteners. Therefore, even if the outer bolts had carried none of the applied load at failure, the inner bolts were less than 100 per cent effective.

Specimen BBP, identical to BB except for a fill plate, behaved in a manner very similar to that described above; but failure occurred at an efficiency which was 3 per cent greater than that of specimen BB. This higher efficiency of BBP may have resulted from the more uniform loading, as seen in Table 3, or from the longer bolt lengths.

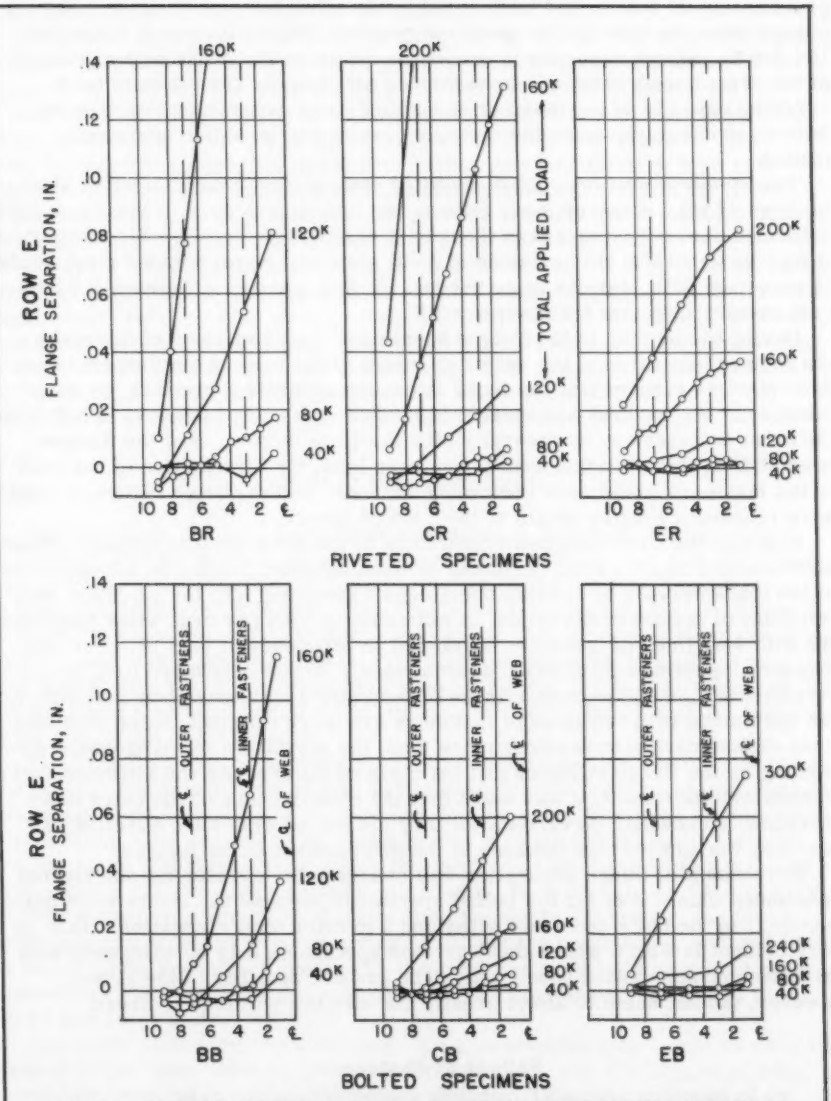


FIG. 6 FLANGE SEPARATION OF RIVETED AND BOLTED MEMBERS WITH FOUR LINES OF FASTENERS

The inner rivets of specimen BR showed a change in elongation after approximately 80,000 lb. had been applied, the outer fasteners began to show a change when the load on the specimen reached 120,000 lb. and at a load of 164,400 lb. failure occurred in one of the rivets as the shank was pulled out of the driven head which had been formed off center. The ultimate load-carrying capacity of the member undoubtedly was reduced by this type of failure, particularly since the specimen seemed to be rather uniformly loaded.

The rivets of specimen BRP exhibited deformation characteristics similar to those of BR. However, near failure, the longer fasteners of specimen BRP deformed somewhat more than those of specimen BR (see Fig. 5). In addition, it may be noted that the inclusion of a fill plate and better formed rivet heads in specimen BRP, despite more unequal loading, gave an efficiency 3.5 per cent greater than that for specimen BR.

During the loading of the B-type specimens with four lines of fasteners, the flanges separated at the webs but rarely at the toes of the flanges. Such deformations require that the inner fasteners elongate more than the outer fasteners. As the load was increased further, the inner fasteners failed while the outer fasteners were generally still far from failure. Had the flanges been stiffer, their bending would have been less, the distribution of the load to the fasteners would have been more uniform, and the outer fasteners would have reached a greater strain at the time of failure.

Although the outer fasteners contribute to the ultimate load capacity, their effectiveness is not great. This can be demonstrated readily by a comparison of the loads carried by various specimens. Specimen AB, for example, with two lines of fasteners developed an efficiency of 74.4 per cent while specimen BB with four lines of fasteners developed an efficiency of only 48.8 per cent. However, specimen BB carried approximately 30 per cent more load than did specimen AB. Similar comparisons of the other specimens show that although the specimens with two lines of fasteners are more efficient, those with four lines of fasteners always carry more load, the magnitude of the increase depending on the flange stiffness and the mode of failure. As the stiffness of the flanges was increased, it was found that the effectiveness of the outer bolts increased somewhat. Nevertheless, they did not become fully effective in carrying the applied load because of the deformation in the flanges.

The amount of flange separation was considerably greater for the riveted specimens than it was for the bolted specimens because the rivets were more ductile than the bolts and could withstand a greater total elongation. This is very evident in Fig. 6 where three riveted specimens may be compared with three comparable bolted specimens. The greater ductility of the rivets, however, did not seem to affect greatly the efficiency of the members.

Failure of Fasteners

Of the 12 riveted members tested, three had rivet head failures which occurred when the shanks withdrew from the heads which had been driven off-center. In two cases, the heads were so badly off center that they were only 1/32 in. thick at one point along the head-shank junction. This condition and the resulting failures can be expected to reduce the efficiencies of the members. In contrast, well formed rivets which were subjected to large bending rarely showed any signs of initiation of failure in the heads. Occasionally

a hairline crack could be seen on the more heavily strained side of a bent rivet although failure occurred in the shank.

Failure by stripping of the nuts developed in 11 of the 16 bolted members. In those 11 members, three failures were in nuts from Manufacturer "N", seven were in nuts from Manufacturer "S" and one involved nuts from both manufacturers. However, even after stripping began, the members, in most cases, continued to carry a large percentage of the maximum load. Care must be taken in comparing specimens having only bolt failures with specimens having nut failures or a combination of failure types since the latter failures may have been premature and thus the ultimate loads and efficiencies lower than would have been the case if only the bolts had failed.

It may be noted that column 14 of Table 3 compares the test efficiencies of identical bolted and riveted specimens. The average efficiency of the riveted specimens was 1 per cent greater than that of the bolted members; however, this difference is not significant. Although the efficiencies of the riveted and bolted connections are approximately the same, it is apparent from column 4 of Table 3 that, in all but one instance, the bolted members were actually stronger (average of 14 per cent) than their riveted counterparts.

Effect of Flange Stiffness

The AB, CB9, and EB9 specimens had identical flange widths and bolt pattern but had different flange thicknesses and therefore different bolts and bolt lengths. A comparison of the deformation of the several flanges is graphically portrayed in Fig. 7. The flange separation at a given load was largest for specimen AB, CB9 had less bending, and EB9 showed almost no bending and only slight separation. The flange separation plots for Row A and Row E (See Fig. 1 for location of rows) for these three specimens and for all the other members tested were similar, indicating fairly uniform bending along the length of the flanges.

The type E and the wider type F specimens were the stiffest considered in the study. The importance of this stiffness is evident from an examination of the fastener deformations in Figs. 4 and 5. The outer fasteners of both specimen EB and EBP showed an increase in strain at almost the same time as did the inner fasteners. In addition, the bending of the flanges was small by comparison with the more flexible specimens. Both Specimens EB and EBP failed by rupturing several fasteners simultaneously. However, the inner fasteners had strained somewhat more than had the outer bolts, and undoubtedly failed first; the impact created by their failure probably caused the outer fasteners to fail and gave the appearance of a simultaneous failure of the inner and outer bolts.

The outer fasteners of the riveted type E specimens also began to elongate almost at the same time as did the inner fasteners, and the flanges of both specimens separated completely before failure. Thus, the stiffness of the flanges, whether riveted or bolted, affected the fastener elongation and flange deformation markedly.

A comparison of the efficiencies shown in Tables 2 and 3 demonstrates that as the stiffness of the flanges increased, the efficiencies also increased. It is apparent also that increasing the thickness of the flanges increased the ultimate load-carrying capacity of the specimens. This fact is shown in Fig. 8 where the efficiencies of the specimens are plotted against a measure of the

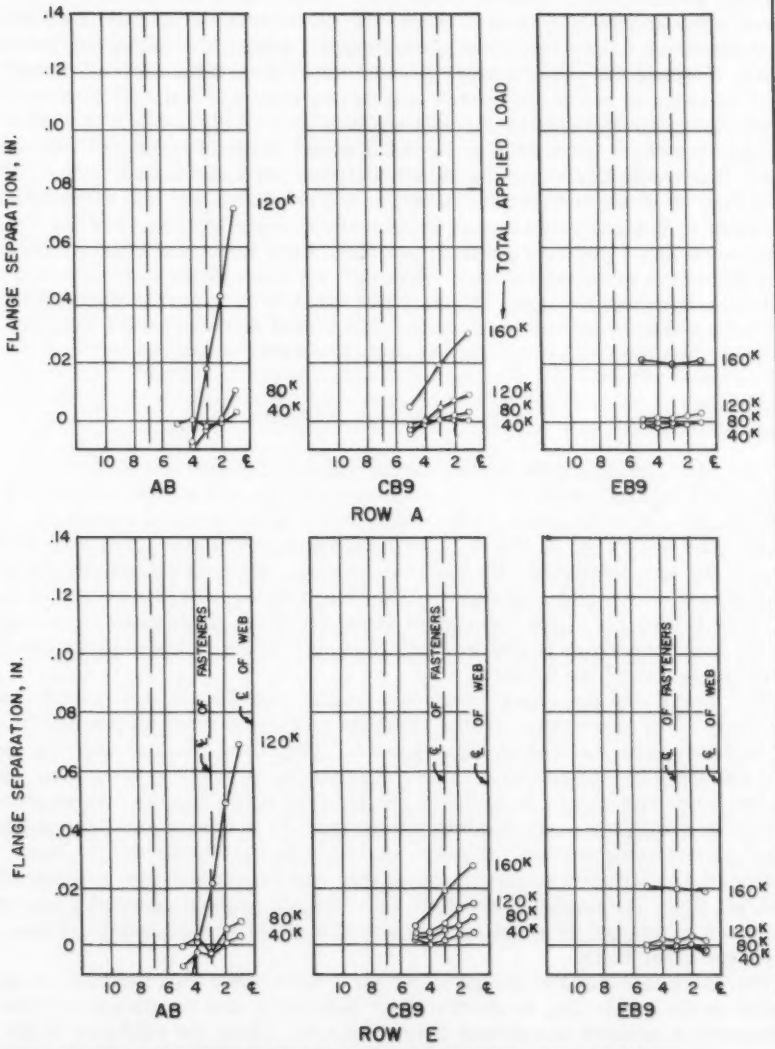


FIG. 7 FLANGE SEPARATION FOR MEMBERS OF VARIOUS FLANGE THICKNESSES

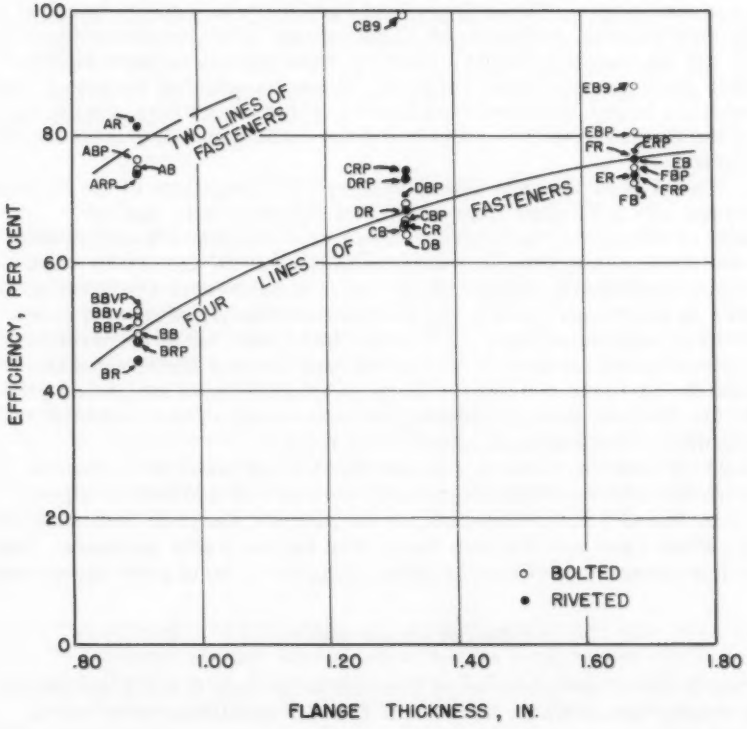


FIG. 8 EFFECT OF FLANGE STIFFNESS ON SPECIMEN EFFICIENCY

flange stiffness. A curve has been drawn through the average values for the specimens with four lines of fasteners and shows the increase in efficiency with an increase in flange thickness. There are insufficient data to draw a comparable curve for the specimens having two lines of fasteners although a similar trend is evident for these members also.

Effect of Grip

Obviously the grip is a function of the flange thickness which in turn affects the stiffness of the members. Since the effects of stiffness have been discussed earlier, the intent here is to examine the grip independently of the stiffness of the flanges. This was accomplished in these tests by inserting a 2 in. fill plate between the flanges of 13 specimens. With the exception of the fill plate and the resulting longer fasteners, these specimens were identical to 13 other specimens having no fill plate. It was thought that because of such a fill plate, the longer fasteners would have greater elongations, the prying action of the flanges would be reduced, and the connection efficiencies might be increased.

Figs. 4 and 5 show that with few exceptions, the elongations of the fasteners in specimens with a fill plate were greater at any given load than the elongations of the shorter fasteners. The several exceptions were probably caused by differences in the mechanical properties of the fasteners. In addition, the inequalities in distribution of load to the fasteners of a given specimen also may have contributed to the unexpected behavior in several cases.

When flange separation plots similar to Figs. 6 and 7 were prepared for the specimens with a fill plate, at any given load, greater flange separations were found for the specimens with a fill plate than for those specimens without the plate. In addition, there did appear, in some cases, to be a reduction in the prying action of the specimens with a fill plate.

Perhaps the best way of examining the effect of the plates is to examine the relative efficiencies of the identical specimens with and without a plate. Column 7 of Tables 2 and 3 show that, on the average, the plate increased the efficiency about 4 per cent for both the riveted and the bolted members. However, such an increase in efficiency does not appear to be of great importance.

Effect of Flange Width

The type D and F specimens were identical to the type C and E specimens in every respect except flange width. The D and F specimens were tested "as rolled" with flange widths of 15.1 in. and 16.7 in., respectively, while the C and E specimens were cut down to a 14 in. flange width.

The deformation characteristics of specimen DR (15.11 in. flange) were similar to those of specimen CR (14.00 in. flange) but the efficiency of specimen DR, shown in column 6 of Table 2, was somewhat higher than that of specimen CR. In a detailed study of the data, it was noted that the rivets used in specimen DR were the most ductile of those tested, and that this may account for the ability of that specimen to reduce its prying action more than was possible in the case of specimen CR. Specimens DRP and CRP, having identical type rivets, deformed in much the same manner, but DRP carried 1.6 per cent less load than did specimen CRP. Thus for the 30 WF 210 specimens with 14 in. and 15.1 in. wide flanges no significant difference exists in

their behavior; and, the small change in edge distance for these comparatively flexible specimens was of no consequence.

The type F specimens, connections with a greater edge distance and stiffer flange than the type D specimens, because of this increased edge distance, exhibited a behavior somewhat different from that shown by the type D specimens. The inner and outer fasteners of the type F specimens elongated more rapidly than those of the narrower type E specimens, and the flanges separated faster, too. However, the ultimate loads of the F specimens were reduced, possibly as a result of the nut failures or rivet head failures that were encountered and also because of the greater prying action possible. A comparison of efficiencies and loads for the 36 WF 300 specimen with 14 in. and 16.7 in. flanges is not entirely valid because the E type specimens all failed in the shank of the fasteners while the F type specimens failed in the rivet head or nut. Such a comparison shows, however, that the effect of the wider flange is to reduce the efficiency about 3-1/2 per cent, although this value may be a little high since all the specimens having other than shank failures were the wider members.

Effect of Initial Tension

Specimens BBV and BBVP were included in the test program to determine whether a lower prestress (0.1 of Elastic Proof Load) in the inner bolts might permit a greater transfer of load to the outer bolts. The inner fasteners did pick up strain faster for specimens BBV and BBVP than they did in the comparable but fully torqued specimens, BB and BBP. However, the outer fasteners behaved in a similar manner for all four specimens. In the early stages of loading, the flanges of the members with the lower prestress in the inner bolts opened up more at the web than did the members with uniform prestress but, when the load reached 160,000 lb., there was little difference in the bending of the flanges. The ultimate loads were somewhat higher with the lower prestress, but the difference was slight (average of about 3%) and might have been less had there been no nut failure in specimen BB.

CONCLUSIONS

1. The efficiency of a tensile joint of the type tested in this investigation is the same whether assembled with bolts furnished to ASTM A-325 specifications or rivets purchased to ASTM A-141 specifications. However, the bolted members carried, on the average, about 14 per cent more load.
2. Members with eight fasteners in four lines are generally less efficient but are stronger than similar members with four fasteners in two lines.
3. The stiffness of the flanges affected the efficiency of the members (a range in efficiency of 30% was obtained for the specimens tested). In one instance it was found that an increase in flange thickness from 0.90 to 1.32 in. provided as great an increase in strength as did the addition of two outer lines of bolts.
4. Inclusion of a fill plate between the flanges of this type of connection will have only a slight effect upon the efficiency (about 4% increase for these tests).

5. Small changes in flange width were of little consequence in these tests.
6. Prestress as low as 0.1 elastic proof load in the bolts in the inner two lines of a specimen with four lines of fasteners will have almost no effect on the strength or efficiency of the member.

ACKNOWLEDGEMENTS

The tests described in this paper were a part of an investigation on riveted and bolted connections being performed in the Civil Engineering Department, under a cooperative agreement between the Engineering Experiment Station of the University of Illinois, the Illinois Division of Highways, the Department of Commerce—Bureau of Public Roads, and the Research Council on Riveted and Bolted Structural Joints. The tests were planned in cooperation with the Project IV Committee of the Research Council on Riveted and Bolted Structural Joints.

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1971 March, 1959 ST 3

Journal of the

STRUCTURAL DIVISION

Proceedings of the American Society of Civil Engineers

BEHAVIOR OF RIVETED AND BOLTED BEAM-TO-COLUMN CONNECTIONS[a]

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SYNOPSIS

The tests reported herein were performed as the first phase of an extensive program designed to study the general behavior of beam-to-column connections assembled with rivets or bolts and included four standard flexible connections. Three of the test specimens were assembled with rivets and high-strength bolts and were tested at different moment-to-shear ratios. The fourth specimen was assembled entirely with rivets so as to provide a study of the effect of the type of fastener upon the behavior of the connections.

A study is made of the moment-rotation characteristics, the moment-resisting capacity, the position of the center of rotation, and the slip and shear deformation of the connections, the deformation of the fasteners, and the separation of the column flanges.

The tests reveal that the connections, although assumed for design purposes to behave as simple supports, actually provide some end restraint and that the restraint provided by the connections was increased slightly by the use of high-strength bolts in lieu of rivets to fasten the connection angles to the column flanges.

INTRODUCTION

In the last few years the ASTM A-325(1) high-strength bolt has become widely accepted as a fastener for the assembly of structural joints.(2) Most

Note: Discussion open until August 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1971 is part of the copyrighted Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. ST 3, March, 1959.

a. Presented at the February 1958 ASCE Convention in Chicago, Ill.

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specifications for the design of steel structures now permit the use of high-strength bolts in lieu of rivets of the same diameter.

One of the most important features of the heat-treated high-strength bolt is the high clamping force developed in the bolt when it is properly tightened. This high tension, generally about twice the maximum developed by rivets as a result of shrinkage during cooling, insures the permanent tightness and soundness of bolted structural joints. In addition, since the clamping force of the bolts does not depend upon shrinkage, it is possible to consistently develop the same clamping force with any length of bolt.

High-strength bolts are well adapted to the field assembly of structural connections for a number of reasons, some of which are as follows: (1) the installation of the bolts requires no heating equipment and thus fire hazards are eliminated, (2) bolting crews can be trained quickly since the equipment required to install the bolts is simple to operate, (3) the bolts are especially economical where it becomes necessary to replace a small number of loose rivets, (4) bolts may be installed with either power or hand operated tools, and (5) field erection costs are often lower when A 325 bolts are used in lieu of field rivets. Consequently, bolts have become competitive with rivets and often replace them for the field assembly of structural connections.

The use of high-strength bolts for the field erection of structures may result in the assembly of many beam-to-column connections with a combination of rivets and bolts. Although data on the behavior of all-riveted beam-to-column connections are available,[3,4,5,6,7] few tests have been performed on connections containing both rivets and bolts. Three of the four specimens tested and reported herein were assembled with a combination of rivets and bolts to provide data on the behavior of such connections. A fourth specimen was assembled entirely with rivets to provide a direct comparison of the moment-rotation characteristics and the general behavior of connections using the two methods of assembly. All of the test specimens were similar to an all-riveted specimen previously tested by Hechtman and Johnston.[6]

The flexible beam-to-column connection, which is designed to resist only vertical loads, is probably the one most frequently used in building construction. To facilitate design, many of these connections have been standardized. These "standard connections" and the corresponding beam reactions for which they may be used are found in the AISC Handbook of Steel Construction.[8] Although assumed to be simple, these connections do provide some restraint at the ends of the beam and, under normal loading conditions, are subjected to both shear and moment.

The tests described in this paper provide information on the effect of several ratios of moment-to-shear upon the general behavior of one type and size of flexible beam-to-column connection. The studies include an evaluation of the moment-rotation characteristics, the moment-resisting capacity, the type and location of failure, the slip and shear deformation, the position of the center of rotation of the connections, the deformation of the column flanges, and the deformation of the fasteners.

Description of Specimens and Tests

Description of Specimens

Each of the test specimens was designed to provide data on the behavior of two identical beam-to-column connections and consisted of two 18WF50

beam stubs 4'-0" in length fastened to the flanges of a 12WF65 column stub 3'-6" in length. The beam stubs were fastened to the column flanges with Standard AISC type K-4[8] connections, each composed of two 6" x 4" x 3/8" x 11-1/2" angles (see Fig. 1).

Three of the test specimens, FK-4A, FK-4B, and FK-4C, were identical in detail except that specimen FK-4C had web stiffeners at the beam supports (3/8 in. thick bar stiffeners were welded to the beam webs and flanges and were located on both sides of the beam webs). The connection angles for these specimens were fastened to the beam webs with rivets of ASTM A-141[9] material and to the column flanges with ASTM A-325 high-strength bolts. The details for Specimen FK-4R were identical to those for the bolted specimens except that the connection angles were riveted to the column flanges as well as the beam webs.

Preparation of Specimens

All of the material for the specimens was furnished by the fabricator from ASTM Designation: A-7[10] stock; the angle material was sheared from one length of material in order that its properties would be uniform from one member to the next.

The rivets in the test specimens and for coupons were furnished by the fabricator from ASTM Designation: A-141 stock. It was specified that all rivets, except those fastening the angles to the column flanges of Specimen FK-4R, be machine driven. Since the connection of the angles to the column flanges would normally be made in the field, the rivets for this part of the connection were hand driven to simulate field conditions.

All of the 2-1/2 in. long high-strength bolts, the nuts, and the washers conformed to the requirements of ASTM Designation: A-325. The grip of the bolts was 15/16 in., the combined thickness of the column flange and the connection angle.

The cutting, milling, punching, and riveting of the test specimens was done in a structural fabricating shop employing standard fabrication techniques. However, the final assembly of Specimens FK-4A, FK-4B, and FK-4C with high-strength bolts through the connection angles and column flanges was completed in the laboratory.

To simulate field conditions for shop painted members, the contact surfaces between the connection angles and column flanges were painted with one coat of red oxide paint. However, before painting, the contact surfaces between the connection angles and column flanges were wire brushed by hand to remove loose mill scale or rust; any grease or foreign material found on the contact surfaces was removed with a solvent. The paint was then applied with a brush and allowed to dry for approximately one week prior to the assembly of the connections. The contact surfaces between the connection angles and beam webs were not painted since this part of the connection is normally made in the shop prior to painting.

The high-strength bolts, which were used in the laboratory to complete the final assembly of the bolted test specimens, were all calibrated in the Laboratory to obtain the nut rotations, torques, and bolt elongations at a bolt tension of 25,600 lb. (the minimum required by the specifications). The elongation readings were then used to obtain the minimum required tension in the bolts during the assembly of the test specimens.

SPEC. NO.	TYPE OF FASTENER BEAM WEB	TYPE OF FASTENER COL. FLG.	a
FK-4A	RIVETS	BOLTS	40"
FK-4B	RIVETS	BOLTS	20"
FK-4C	RIVETS	BOLTS	10"
FK-4R	RIVETS	RIVETS	40"

NOTE: ALL FASTENERS 3/4"; ALL HOLES PUNCHED 13/16"

(N) SLIP, DEFORMATION AND ROTATION DIALS

DEFLECTION WIRE

STIFFENERS FOR FK-4C LOCATED AT THE SUPPORTS

FIG. 1 GENERAL DETAILS OF SPECIMENS AND INSTRUMENTATION

The bolts were tightened with a manual torque wrench to an elongation corresponding to only the minimum required bolt tension so that the test connections would have the minimum rigidity of bolts which are installed in accordance with the specifications for bolting. For the forty-eight bolts used, the average elongation at the required minimum bolt tension was 0.0046 in., the average torque was 300 lb. ft., and the average nut rotation was 6/10 of a turn. The rotation of the nut was measured from a "finger tight" position, achieved by turning the nut as far as possible with the fingers while the material to be joined was properly fitted-up with adjacent fitting-up bolts.

For the bolting of the test specimens the position of the column was reversed from that to be used in the testing machine so that the weight of the beams would produce bearing against the bolts in the same direction as would the beam reactions during the tests. This procedure produced bearing at some, if not all, of the bolts and simulated field conditions whereby the weight of the beam causes it to bear on the bolts before they are tightened.

Properties of Materials

One coupon was taken from each leg of the 6" x 4" x 3/8" angle and tested in the "as-rolled" condition. The mechanical properties of the angle material, as determined from these coupon tests, are listed in Table 1. It can be seen that the angle material satisfied the requirements of ASTM Designation: A7-55T,[10] but that the material from the 6 in. angle leg had a slightly higher yield point and ultimate strength than the coupon material from the 4 in. angle leg.

Coupons were machined from the bolts and the undriven rivets to dimensions proportional to the Standard "0.505-in. Coupon" as provided by ASTM Designation: E8-54T.[11] These coupons had a diameter of 0.250 in. and a 1.00 in. gage length. The mechanical properties of the rivets and bolts are listed in Tables 2 and 3 respectively.

Full-size bolts were tested in the as-received condition to determine the proof load* and the ultimate strength of the bolts. The bolts were first loaded to the required proof load, 28,400 lb. for 3/4 in. diameter bolts, and then unloaded to check for permanent set. The bolts were next tested to failure either in direct tension with parallel surfaces or in tension with a 10^{0} wedge under the head of the bolt. The average ultimate strength of the bolts was 46,700 lb.; thus they met the requirements of ASTM Designation: A-325-55T.[1]

Instrumentation and Equipment

All of the test specimens had similar instrumentation; however some minor changes were made as the tests progressed. This instrumentation included mechanical dials, SR-4 electric resistance strain gages, scales for the determination of deflections, extensometers, and a qualitative visual indicator of the location and extent of yielding.

*The proof load is that load, as required by ASTM Designation: A-325-55T,[1] to which a bolt must be subjected without experiencing any permanent set. A tolerance of ± 0.0005 in. is allowed as the difference between the measurements made before and after the loading.

FIG. 1 GENERAL DETAILS OF SPECIMENS AND INSTRUMENTS

TABLE 1

MECHANICAL PROPERTIES OF ANGLE MATERIAL

Standard 8 in. Gage Length

Origin of Coupon*	Elongation in 8 in. per cent	Reduction in Area per cent	Yield Strength** Upper ksi	Yield Strength** Lower ksi	Ultimate Strength ksi
4" leg	28.1	53.9	39.3	37.7	62.5
6" leg	31.3	51.0	41.3	38.4	65.4
Average	29.7	52.5	40.3	38.1	64.5

*All angles were 6" x 4" x 3/8" x 11-1/2".

**Determined by "drop of beam."

Mechanical-dials were used to obtain rotations and movements in the tests (See Figs. 1 and 2). These dials were used to measure: (1) the rotation of the end of the beam relative to the column,—dials 1 and 2; (2) the angular movement of the beam relative to the connection angles,—dials 3 and 4; (3) the vertical movement of the connection angles relative to the column flanges,—dials 7 and 8; (4) the vertical movement of the beam relative to the connection angles,—dial No. 6; and (5) the vertical movement of the beam relative to the column flange,—dial 5.

The SR-4 electric resistance strain gages were used on Specimen FK-4A to indicate the relative distribution of the column load to the two beam reactions. The gages were mounted on the flanges of each beam midway between the column face and the beam support. Since the strain gage readings for Specimen FK-4A indicated an equal distribution of the column load to the beam reactions, the use of the gages was discontinued in subsequent tests.

The deflection scales, graduated in 0.05 in. divisions, were mounted on the edges of the column flanges as shown in Fig. 1. These scales were read with reference to a fine wire stretched between two rods which were mounted over the supports at the mid-depth of the beams. As the column was loaded, the scales moved downward relative to the wire which remained in a horizontal

TABLE 2

MECHANICAL PROPERTIES OF RIVET MATERIAL

Standard 0.252 in. Coupons

Normal Size of Rivet	Elongation in 1 in. per cent	Reduction in Area per cent	Yield Strength* ksi	Ultimate Strength ksi
3/4 x 2-1/4	35.5	65.0	35.6	62.2
3/4 x 2-1/4	35.0	62.0	37.3	64.1
Average	35.3	63.5	36.5	63.2

*Determined by "drop of beam."

TABLE 3

MECHANICAL PROPERTIES OF HIGH-STRENGTH BOLT MATERIAL

Standard 0.252 in. Coupons

Bolt Length in.	Elongation in 1 in., per cent	Reduction in Area, per cent	Yield Strength* ksi	Ultimate Strength ksi
2-1/4	18.0	53.5	87.9	125.7
2-1/4	15.0	53.3	--	147.4
Average	16.5	53.4	87.9	136.6

*Yield strength determined from 0.2 per cent offset.

position. Thus, the movement of the scales relative to the wire represented the deflection of the center of the beams at the supports relative to the column.

Two types of extensometers were used in the tests, one to determine the deformation of the fasteners through the connection angles and the column flanges and the other to measure the separation of the column flanges. (The points at which the flange separation was measured are adjacent to the numbers on the inside of the column flange in Fig. 5).

Results and Analysis of Tests

Description of Tests

Three of the test specimens, FK-4A, FK-4B, and FK-4R, were tested to maximum load carrying capacity; however, the fourth specimen, FK-4C, could be loaded only to 300,000 lb., the maximum capacity of the testing machine. In the tests, the specimens were subjected to a number of load increments; after each increment of loading a series of measurements were taken and a visual inspection of the various components was made.

The distance from the column faces to the beam supports, shown as "a" in Fig. 1, was maintained constant during each test. However, the bearing plates which were fastened to the beam flanges moved outward relative to the supports. This movement can best be understood by visualizing a line connecting the bearing plate with the center of rotation of the connection (assumed to be located at approximately mid-depth of the beam). Then, as the column deflects downward, this line rotates about the center of rotation of the connection and the horizontal distance between the bearing plate and the column face increases. Since it was considered desirable to maintain a constant distance between the column faces and the beam supports during the tests, the bearing plates moved outward by slipping and introduced, between the bearing plates and the supports, a friction force which must be considered in determining the actual moment on the connections.

The beam reaction which acted normal to the beam flange and the friction force which acted parallel to the beam flange were included in the determination of the connection moments. However, the magnitude of the effect of the

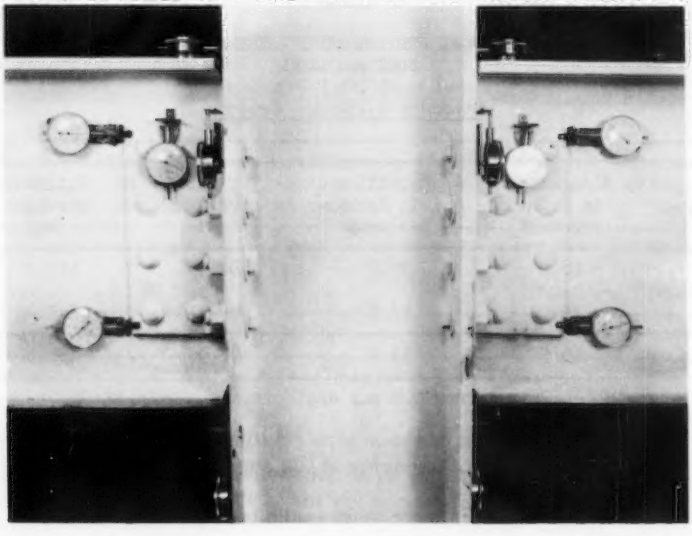

FIG. 2 INSTRUMENTATION FOR ROTATION AND SLIP MEASUREMENTS

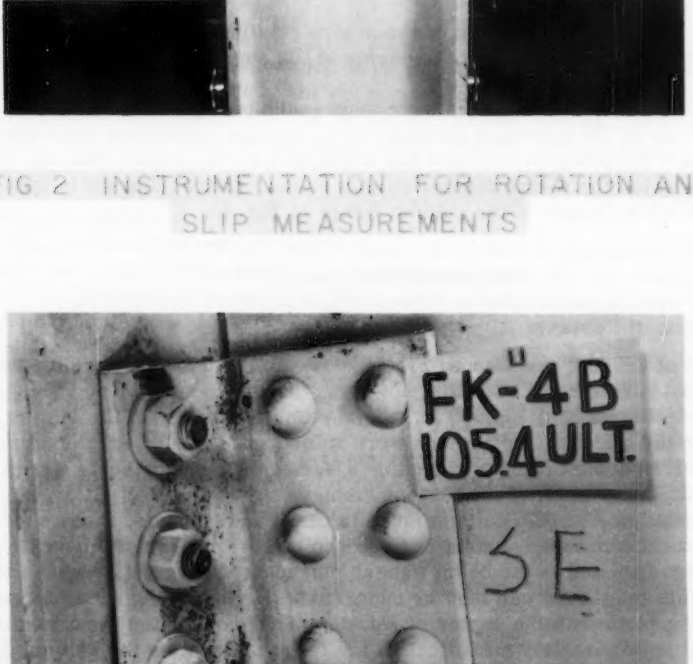

FIG. 3 YIELDING AND TEARING OF CONNECTION ANGLES OF SPECIMEN FK-4B

friction force upon the connection moment depended largely upon the span length. Specimen FK-4C, a short span member, required a greater beam reaction to produce a given connection moment than did the other members and, as a result, received the greatest moment correction for the frictional force.

The total connection moments, including the effect of the beam reactions and the friction force, have been noted in the figures of this paper as "adjusted connection moments" to differentiate them from the moments based only on the vertical beam reactions. All references in the text to connection moment, unless otherwise identified, pertain to the "adjusted connection moment" as defined above. With the exception of the data of Fig. 7, all of the test data have been analyzed and are discussed in terms of the adjusted connection moment.

Discussion of Test Results

Since all the connections were identical in detail except for the use of rivets in lieu of bolts through the angles and column flanges of Specimen FK-4R, it was found that the general behavior of all of the specimens was quite similar. Only the behavior of Specimen FK-4A is presented below; a comparison of the test results for all of the other specimens is made later in the text.

Specimen FK-4A was the first specimen tested and, in order to assure the determination of all important characteristics of its behavior, more observations and instrumentation readings were taken than in subsequent tests. However, the principal measurements in the tests were the rotation measurements and these were obtained in each instance to provide the desired moment-rotation relationships. The average moment-rotation curve for Specimen FK-4A, as well as the other specimens, is presented in Fig. 6.

At a connection moment of 260 in. kips, the heels of the connection angles on the tension side of FK-4A began to separate from the column flanges. However, since the fasteners on the tension side of the connection had experienced no change in elongation at this moment, the separation or deformation of the angles occurred only between the two lines of fasteners connecting the angles to the column flanges; the toes of the angles remained in contact with the column.

At a moment of 225 in. kips, yield lines began to appear at the fillet of the angles on the compression side of the connections and was indicative of the high bearing pressure of the heels of the angles against the column flanges. At a moment of 285 in. kips similar yield lines began to appear at the fillets of the column web opposite the heels of the connection angles, further indicating the presence of high bearing pressures. Yield lines of this type, at a later stage of the test of Specimen FK-4R, are readily visible in Fig. 4.

The connection angles of FK-4A were found to have separated from the column flanges a distance of 0.50 in. at a moment of 362 in. kips and to have started yielding at the fillets on the tension side at a moment of 450 in. kips. Then, as the beams continued to rotate relative to the column faces, the beam flanges in compression came into bearing against the column flanges. Shortly after the beam flanges had made contact with the column flanges, yield lines appeared at the fillets of the column opposite the points of contact and indicated the concentration of pressure in the column web.

At a moment of 585 in. kips, one of the east and one of the west connection angles began to tear at the fillet of the angles on the tension side of the

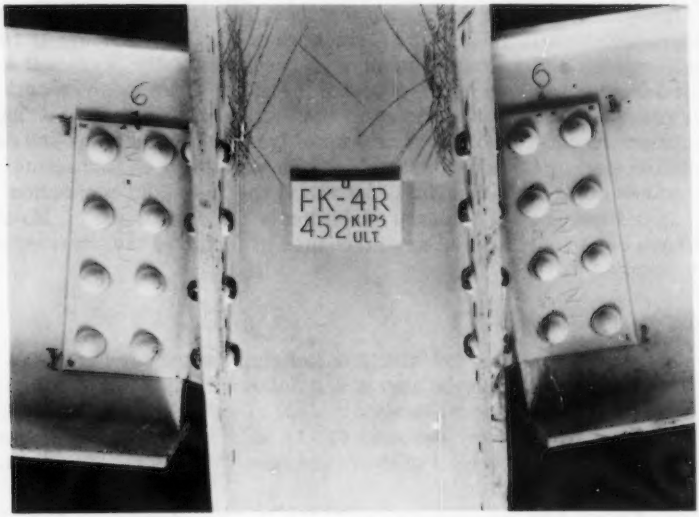

FIG. 4 YIELDING OF COLUMN WEB ON COMPRESSION SIDE OF CONNECTIONS

FIG. 5 YIELDING OF ANGLES AND BEAMS OF SPECIMEN FK-4C

connections. The loading of the specimen was continued beyond the load at which this tearing began and until a maximum load was reached.

A general summary of the behavior of the specimens and the test results is presented in the following tabulation.

SUMMARY OF TEST RESULTS

Adjusted Connection Moment (in. kips)				
FK-4A	FK-4B	FK-4C	FK-4R	Condition
225	230	*	235	Yielding of angles at fillet on compression side
285	270	270	290	Yielding of column at web fillet on compression side
450	360	490	360	Yielding of angles at fillet on tension side
362 0.50"	380 0.60"	390 0.45"	395 0.60"	Moment at large separation of angles from column flanges and magnitude of separation
585	450	**	450	Tearing of angles
46.86	105.4	300 +	45.2	Maximum load resisted by specimens, kips
40	20	10	40	Distance from column face to beam support, in.

* Observation not recorded.

** No tearing of angles although maximum capacity of machine, 300,000 lb. was reached.

It is of interest to note that, in general, the various connections behaved in a similar manner and that the various conditions such as yielding, tearing of angles, etc., generally occurred at approximately the same moments in each of the members. Thus, it would appear that the variation in span had no consistent or marked effect on the behavior. However, it may be seen that the angles of the bolted specimen FK-4A tore at a somewhat greater moment and failed at a slightly greater load than did the angles of the riveted specimen FK-4R, indicating that the bolts might provide a little increase in strength to failure.

During all of the tests, the separation of the connection angles from the column flanges caused the angle legs which were fastened to the column flanges to pull in toward the beam webs. As a result, the legs of the angles were soon bearing against the fasteners and causing deformations of the bolt holes and bending of the fasteners. As the bolted connection angles continued to separate from the column flanges, the flat heat-treated washers between the nuts and angles began to bend in the same direction as the deformed angles but, although these washers experienced severe bending, only one washer cracked.

With the exception of Specimen FK-4C, which was loaded to the maximum capacity of the testing machine without failure, all of the connections experienced a tearing of the angles on the tension side of the connections. Although the connections were subjected to both moment and shear, the tearing of the

angles resulted primarily from the bending moment. This can be seen readily from a comparison of the behavior of Specimen FK-4C with that of Specimens FK-4A, FK-4B, and FK-4R. Specimen FK-4C resisted a vertical beam reaction of 150,000 lb., almost three times the allowable beam reaction for this type of connection, without experiencing a tearing of the angles. However, the angle of Specimens FK-4A, FK-4B, and FK-4R tore at beam reactions of 16,000, 29,500 and 12,900 lb., respectively.

It is of interest to note that all of the angles which experienced tearing had a slight depression about 1 in. in length in the fillet, as shown in Fig. 3 (see arrow). These depressions, a result of the shearing operations, evidently weakened the angles sufficiently to initiate tearing under the tensile loadings reported. The angles which did not tear had a similar depression; however, because of the manner in which the two angles of a connection are assembled, the depressions in the untorn angles were located on the compression side of the connections.

Moment-Rotation Characteristics

The term rotation, as used in this report, refers to the rotation of the end of a beam relative to the column. This rotation was measured with two mechanical dials mounted at the end of the beam flanges and bearing against the center of the column flange.

The deformation in the angles at the tension side of the connection angles was found to provide the largest component in the rotation at the ends of the beams. Nevertheless, there were a number of other factors which also contributed to the rotation, including: the extension of the fasteners which connected the angles to the column flange, the deformation of the column flanges, and the angular movement between the beam web and the connection angles.

Two moment-rotation curves were obtained for each test specimen except Specimen FK-4A. From these data it was evident that there was little difference between the moment-rotation characteristics of the two connections for a given specimen and that all of the curves were of the same general shape (See Fig. 6). At low loads (moments up to 200 in. kips) the rotation of the beams was approximately proportional to the connection moment. Then, as the connection angles began to yield, the rotation increased in a non-linear manner with respect to the applied moment; however, as the moment was increased still further, the rotation assumed a new linear variation with moment and continued this behavior up to the maximum moment for which the rotation readings were taken.

The average moment-rotation curves for the various specimens and a similar all-riveted specimen tested at Lehigh University[6] are shown in Fig. 6 and clearly demonstrate that the behavior of the various members was quite similar. A comparison of the moment-rotation curves of Specimens FK-4A (bolted) and FK-4R (riveted), members fabricated of the same angles, beams, and column materials, indicates a slightly greater resistance to rotation for Specimen FK-4A. This slight increase in resistance to rotation may have resulted from the stiffening effect of the heat treated washers and from the greater resistance of the bolts to bending and elongation.

A comparison of the moment-rotation curves for the two riveted specimens, FK-4R and the Lehigh specimen, indicates a greater resistance to rotation for the Lehigh specimen. However, as pointed out in the report of the Lehigh tests,[6] the rotation, produced by a given moment, may vary by as

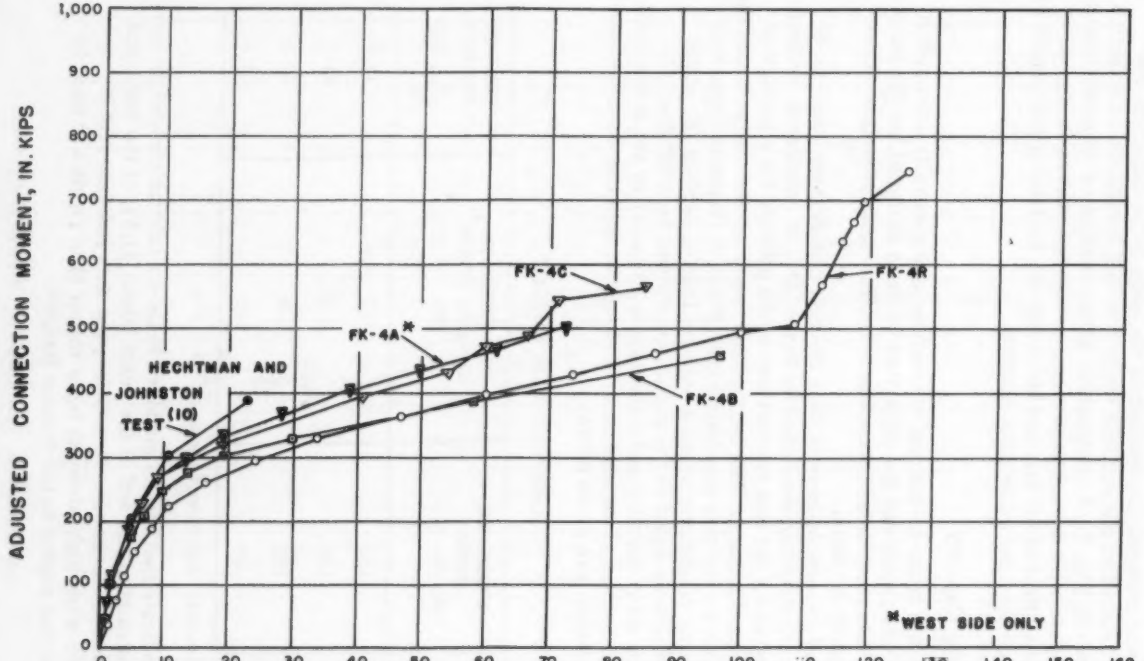

FIG. 6 COMPARISON OF MOMENT-ROTATION CURVES FOR RIVETED AND BOLTED CONNECTIONS

much as 20 to 50 per cent for identical specimens supplied by several fabricators. The difference of the moment-rotation data for these two specimens may also be due, in part, to the use of different rotation instrumentation in the two tests. The rotation of the beams of the Lehigh specimen was measured with level bars,[6] whereas mechanical dials were used for Specimen FK-4R.

The relationship between the rotation of the ends of the beams at the column and the moment of the vertical beam reaction (unadjusted connection moment) is shown in Fig. 7. A comparison of the unadjusted moment-rotation curves in this figure with the adjusted moment-rotation curves in Fig. 6 shows the marked effect of the friction force and the horizontal component of the beam reaction upon the connection moment.

Moment-Resisting Capacity

Standard flexible beam-to-column connections are generally assumed to behave as simple supports and the supported beam is designed for the simple-beam moment. However, these connections do offer some resistance to rotation at the ends of the beam.

With the moment-rotation curves from the test connections (see Fig. 6) and a graphical construction developed by Batho,[5] it is possible to evaluate the end restraint which the test connections would provide for a particular beam size, span, and loading. The beam size chosen for the analysis was the same as that used for the test specimens (18WF50). A laterally supported span of 11.2 ft. was chosen for which a uniform load of 9400 lb./ft. would produce beam reactions equal to the maximum allowed by AISC Specifications[8] for the K4 connection and a maximum stress of 20 ksi at the center of the span. The results of the analysis, in which the columns were assumed fixed against rotation, are given below.

MOMENT-RESISTING CAPACITY

SPECIMEN	ACTUAL END MOMENT M, in. kips	REDUCTION IN MID-SPAN STRESS* ksi	PER CENT RIGIDITY $\frac{M}{M_F} \times 100$
FK-4A	150	1.7	12.6
FK-4B	136	1.5	11.5
FK-4C	125	1.4	10.5
FK-4R	90	1.0	7.6

*Based upon allowable mid-span stress of 20.0 ksi.

These data indicate that the K4 standard beam-to-column connection provides partial restraint at the end of the beam (about 11.5% of the fixed-end moment for bolted connections). In addition, the data indicate that the restraint is greater when high-strength bolts rather than rivets are used to fasten the connection angles to the column flanges.

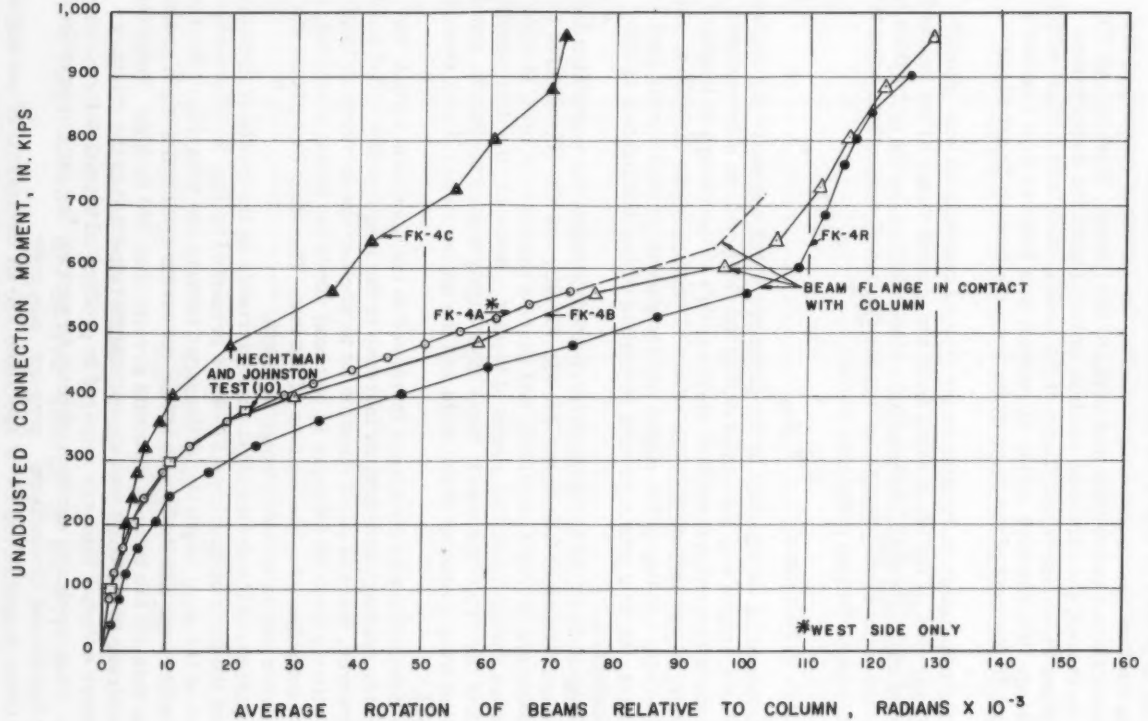

FIG. 7 UNADJUSTED MOMENT - ROTATION CURVES

Effective Center of Rotation

The readings from the rotation dials (dials 1 and 2, Fig. 1) were used to determine the locus of the points about which the connections seemed to rotate during the tests. The location of these points, the distance from the tension flange of the beams to the center of rotation of the connections, was studied and is plotted in Fig. 8 for the connection moments.

During the first few increments of loading the center of rotation of the bolted connections was near the mid-depth of the beams. However, as the moment increased, the center of rotation moved toward the compression flange. When the compression flange of the beams came into contact with the column flange, the effective center of rotation was found to have reached a point about 70 per cent of the beam depth from the tension flange of the beams.

The effective center of rotation of the riveted connections was approximately 62 per cent of the beam depth from the tension flange of the beams for the first loading. However, the position of the effective center of rotation had moved to about the same location as that for the bolted connections when the beam and column flanges came into contact.

Deformation of Fasteners

The deformation (elongation) of the fasteners which connected the angles to the column flanges was measured during the tests with an extensometer. Although the deformation discussed herein refers to the change in length of the fastener, it may include also the effects of bending of the fasteners, which results from the bending of the angles and the pulling-in of the angle legs. Nevertheless, the data give a general indication of the deformation in the various fasteners.

The elongations of the bolts which were used to fasten the connection angles to the column flanges of Specimens FK-4A, FK-4B, and FK-4C indicated that a slight decrease in length occurred, and hence a decrease in load, in all of the fasteners during the initial stages of loading. However, the decrease in the length of the "Top" fasteners, those most highly loaded in tension, probably resulted from a bending of the bolts and the angles rather than a reduction in bolt load.

As the connection moments were increased to the maximum values, the extensometer readings indicated an increase in the length of the "Top" tension bolts. However, there is some question as to whether the increase in length resulted entirely from an increase in bolt load above the initial prestress, the bending of the bolts, or a combination of the two. The other three bolts did not experience any noticeable change in length.

A comparison of the deformation characteristics of the rivets of Specimen FK-4R and the bolts of Specimen FK-4A is presented in Fig. 9. These curves reveal one of the most important differences between the behavior of the connections with rivets and those with bolts: the rivets exhibited a much greater elongation than did the bolts at moments greater than 200 in. kips. Since the initial prestress in the bolts was approximately twice that in the rivets, a much greater external load was required to produce additional deformation in the bolts. This, no doubt, accounts at least in part for the greater rotation noted for the riveted connections.

In the latter stages of testing of Specimen FK-4R the tearing of the angles weakened them and caused a slightly greater portion of the load to be resisted

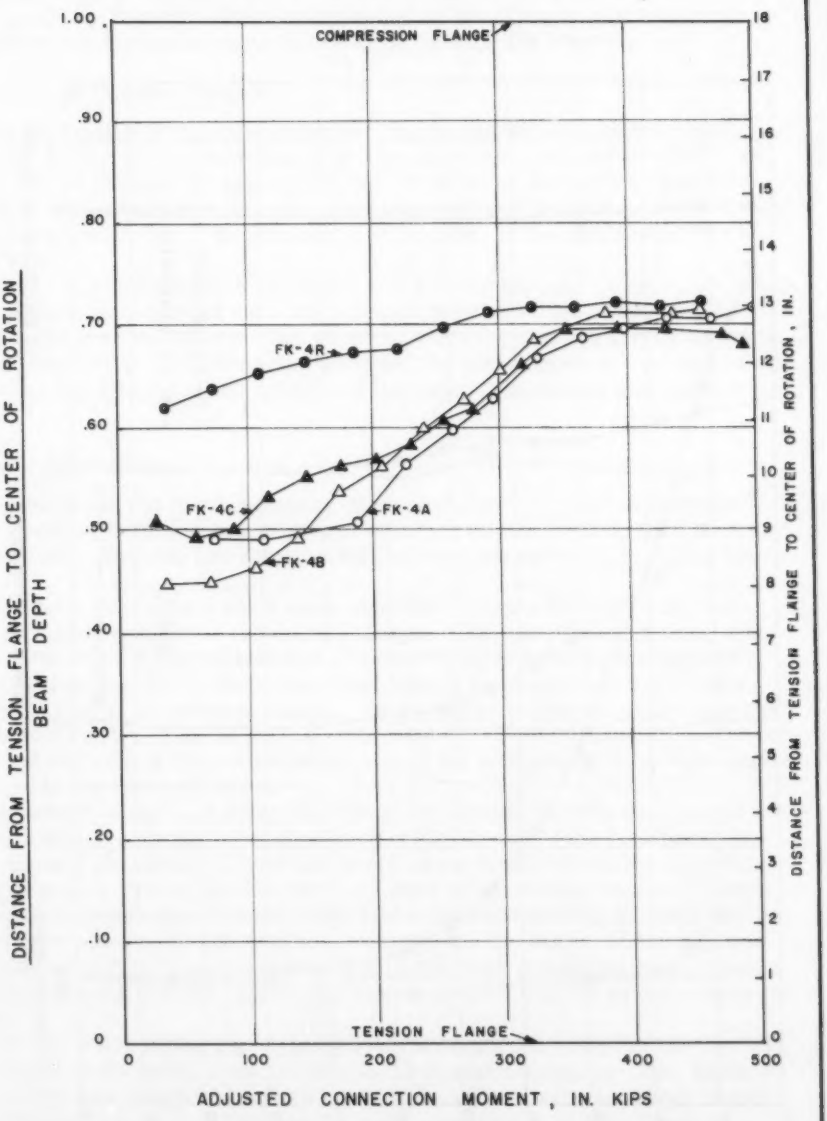

FIG. 8 POSITION OF EFFECTIVE CENTER OF ROTATION DURING TESTS

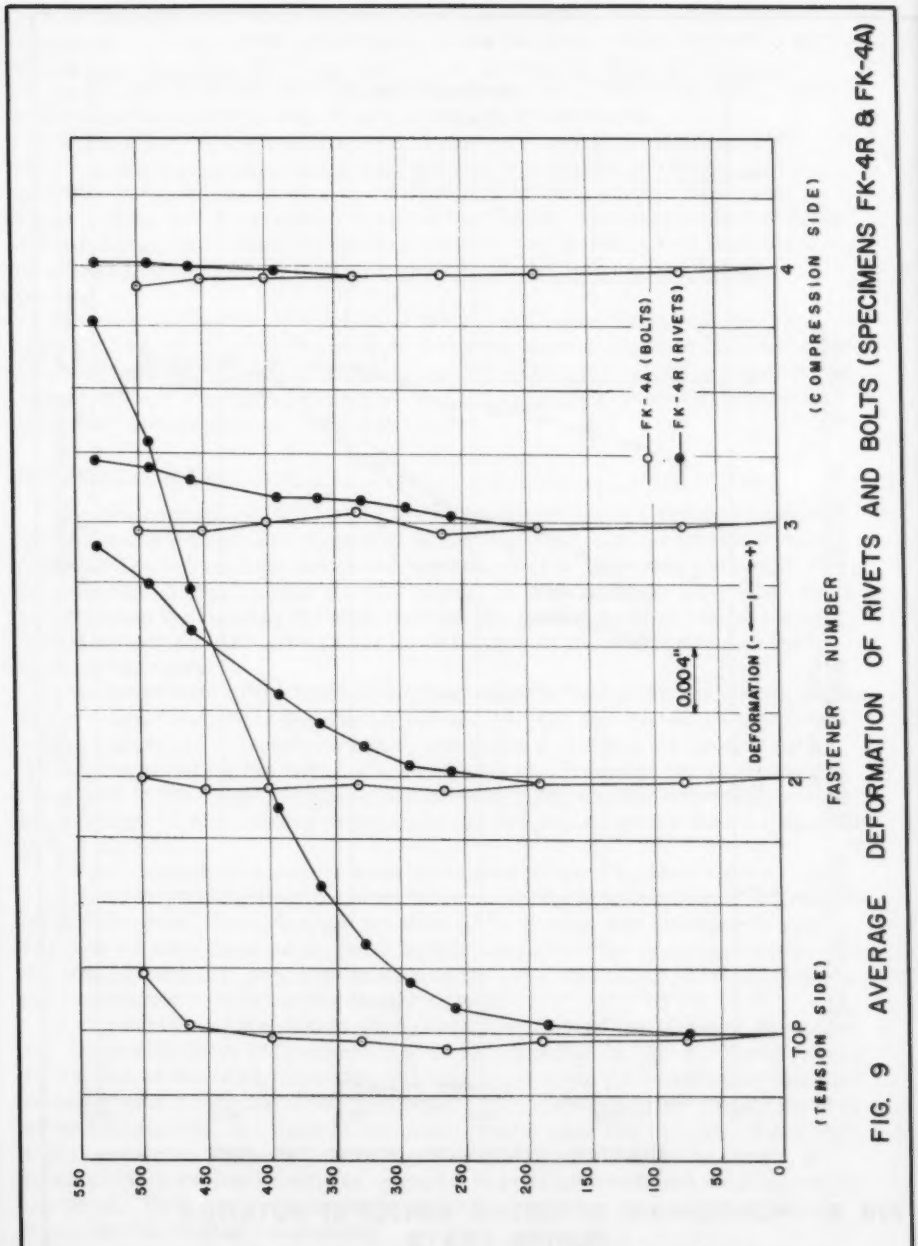

FIG. 9 AVERAGE DEFORMATION OF RIVETS AND BOLTS (SPECIMENS FK-4R & FK-4A)

by the "Top" rivet of the untorn angle. Then, as tearing progressed, a greater portion of the tensile force in the torn angle was resisted by the No. 2 rivet, as indicated by an increase in its deformation. The deformation characteristics of the No. 3 and 4 rivets for both the torn and untorn angles were similar. However, after the beam and column flanges came into contact, all of the fasteners began to resist (in tension) the external load.

Separation of Column Flanges

The separation of the column flanges was measured with an extensometer (dial No. 9, Fig. 1) at points spaced at 3 in. intervals along the centerline of the fasteners through the column flanges. The flange separation measurements on the two sides of the web, were averaged and the results plotted from the column centerline in the manner shown in Fig. 10 for Specimens FK-4C and FK-4R.

All of the column flanges deformed in a similar fashion. That is, the maximum separation occurred near the gage points between the "Top" and the No. 2 fasteners, and the direction of the movement of the flanges reversed at a point near the No. 4 fasteners. However, the contribution of this separation of the column flanges to the rotation of the ends of the beams was relatively small.

Slip and Deformation

In addition to the instrumentation discussed above, the test specimens were provided with instrumentation to measure slip and deformation in the connections. The slip and deformation dials are numbered 5, 6, 7, and 8 in Fig. 1.

The dials numbered 7 and 8 were selected to obtain the slip of the connection angles relative to the column flanges. But, the data from these dials were difficult to interpret because of the error introduced by the angular movement or rotation of the outstanding legs of the connection angles in a plane parallel to the column flanges. This angular movement of the legs resulted principally from the pulling-in of the legs at the tension side of the connections and affects the movement or slip of the outstanding legs of the angles relative to the column flanges.

No sudden changes or irregularities were noted in the slip readings of any of the bolted specimens, even though the shear on one of the members, FK-4C, reached 100,000 lb. (28,400 psi shear on the bolts) before the slip dials were removed. Thus, the deformations were primarily the result of elastic and plastic deformations of the connections rather than slip, although the holes provided the normal 1/16 in. clearance for the bolts. Major slip was probably absent because the bolted specimens were fabricated with at least a part of the bolts initially in bearing, just as would normally be the case in the field.

The No. 6 dials (See Fig. 1) were used to obtain a measure of the vertical movement of the beam webs relative to the connection angles. But, again, no major slip was observed because the beam reactions were not large enough to overcome the friction between the connection angles and the beam webs, and the short rivets probably filled the holes, allowing little or no clearance for slipping.

The vertical movements of the beams relative to the column flanges were obtained with the No. 5 dials (See Fig. 1). These movements, when corrected

FIG. 9 AVERAGE DEFORMATION OF RIVETS AND BOLTS (SPECIMENS FK-4R & FK-4A)

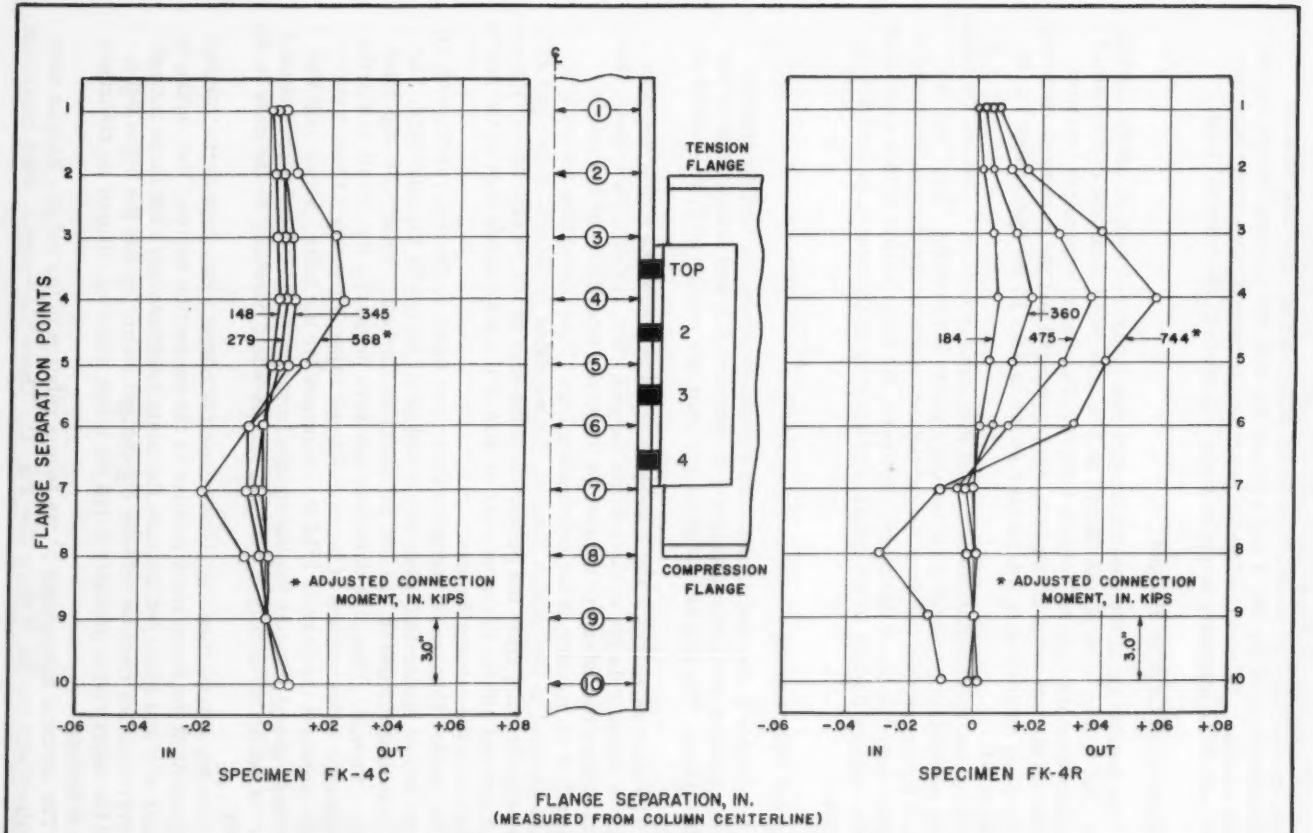

FIG. 10 AVERAGE SEPARATION OF COLUMN FLANGES

for the effect of the rotation of the ends of the beams, provided relationships of the same general type as those obtained from dials 7 and 8, the slip of the connection angles relative to the column. However, this should be expected since no slipping was evident between the angles and column flanges or between the beam webs and angles.

Summary of Results

The results of the tests reported herein may be summarized briefly as follows:

1. The standard (flexible) K-4 connection, which is assumed for design purposes to behave as a simple connection, provides some restraint against rotation (about 10% fixity with an 18WF50 at working stresses).
2. The moment-rotation characteristics of the riveted and the bolted connections were very similar. The three bolted connections, although subjected to loads producing different moment-shear ratios, all gave the same general relationship between moment and rotation.
3. The rotation of the connections resulted primarily from the deformation of the angles. The elongation of the fasteners, the deformation of the column and the slip between the connected parts had only a negligible effect on the rotation.
4. The use of high-strength bolts in lieu of rivets to fasten the connection angles to the column flanges increased slightly the restraint provided by the test connections.
5. The point about which the connections seemed to rotate during the tests moved from an initial position near the mid-depth of the beam toward the compression side of the connections.
6. The test connections were subjected to bending moment and shear. However, the deformation and subsequent tearing failure of the connection angles resulted primarily from the bending moment; the effects of shear deformation were small.
7. The test connections failed by tearing of the angles without rupture of the fasteners, even though, in one case, the applied load reached a value nearly three times as great as the allowable design load for the connections. This tearing initiated at depressions which had been created during the shearing operations.
8. The rivets, which fastened the connection angles to the column flanges, experienced permanent elongation at a lower connection moment than did the high-strength bolts.

FIG. 10 AVERAGE SEPARATION OF COLUMN FLANGES
FLANGE SEPARATION, IN.
(MEASURED FROM COLUMN CENTERLINE)
SPECIMEN PK-4C

ACKNOWLEDGMENTS

The tests described in this paper are a part of an investigation on riveted and bolted joints being carried on in the Civil Engineering Department of the Engineering Experiment Station at the University of Illinois in cooperation with the Illinois Division of Highways, the Department of Commerce—Bureau of Public Roads, and the Research Council on Riveted and Bolted Structural Joints. The tests were planned in cooperation with the Project IV Committee of the Research Council.

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Journal of the
STRUCTURAL DIVISION
Proceedings of the American Society of Civil Engineers

FATIGUE TESTS OF JOINTS OF HIGH STRENGTH STEELS

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SYNOPSIS

A condensed summary is presented of a program of fatigue testing, in direct tension, of joints made from structural steels having various properties. Rivets and high strength (A 325) bolts were used as fasteners, but the middle plate of the double-lap specimens was always critical in fatigue. It is concluded that clamping force was the most important variable governing the fatigue strength of the joints tested. In the riveted joints, the variations of clamping force subordinated whatever superiority one steel has over another. Joints having high clamping force have high fatigue strength; the joints connected with high strength bolts show much higher fatigue strength than those connected with rivets, even approaching the yield points of some of the steels.

Purpose of Tests

From the outset, the leading objective of the program has been to determine how much increase in load is allowable when steels of higher nominal strength than the usual A7[2] steel are used in structures (or parts thereof) in which fatigue strength is the limit of usefulness. The many tests with static loading, and those employing the weldable structural steel A373 and the A7 steels were made for comparative purposes. Initially the test program employed as fasteners only rivets, of two different materials: Carbon steel, A141, and a manganese steel, A195. Somewhat later, tests were begun using as the fasteners high strength bolts, specification A325. The purpose in varying the fasteners was to discover what effects their variation would have on

Note: Discussion open until August 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1972 is part of the copyrighted Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. ST 3, March, 1959.

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2. A7, and similar designations hereinafter, refer to standard specifications of the American Society for Testing Materials.

the fatigue behavior of the plate material. As the second, or supporting purpose of the entire program, it was hoped to elucidate the effects on fatigue life of several variables, including joint arrangement, properties of the main plate materials and of the fasteners, and intensity of stress.

Description of Test Programs

The Committee has conducted two test programs, of which the second is specifically the subject of this paper. However, for background and comparison some data will be included from Program I. In both programs, the test method has been to subject specimens prepared from rolled plates to a cycle of direct tensile stress, varying in each cycle from zero to a present maximum: this maximum stress is a principal variable. All specimens of each steel were taken from a single plate throughout both programs. Steel A, for example, designates steel from only one plate. Different steels to the same specification were obtained from different mills. Table 1 gives the physical properties of the steels tested. Mill scale was left on. In the first program, sixteen different steels were tested in the form of a plate with a hole, and the effect of the stress concentration due to the hole was assessed by comparison with results on similar plate specimens without the hole. A few more than one hundred tests of plates with a hole were made.

In Program II, the test specimens were double lap joints, with the thicknesses of the outer and middle plates chosen to make the middle plate always critical. The fasteners, whether rivets or bolts, were not intended to fail in fatigue, and they did not. There were three patterns of joints, as shown in Fig. 1; Pattern A with two fasteners in a line, Pattern B with four in a line, and Pattern C with four in a rectangle. It may be noted that Pattern C is a doubling in width of Pattern A: the ratio of net to gross area, 74.5 per cent, and the ratio of gage to diameter of fastener (g/d ratio), 4.25, are the same for these two patterns. The ratios for Pattern B are higher: net to gross area, 85 per cent; and g/d, 7.33. Seven steels from the first program, plus three others, were tested, although not all of them with all combinations of pattern and fastener. A total of 210 specimens were tested in fatigue under Program II; and static load tests were made of each kind of specimen tested in fatigue.

All of the fatigue tests were performed on the direct-tension machine of the Northwestern Technological Institute, Evanston, Illinois. This is a dual machine of Wilson type, and contains two independent testing units mounted in a single frame. In each unit, a fixed and a movable pull head are so mounted that the load may be maintained concentric with the axis of the specimen. A walking beam transmits the motion from a gear train and eccentric to the movable head. Load capacity is 250,000 pounds and the operating speed is approximately 170 cycles per minute. Load measurement was by means of a proving ring.

Holes in all specimens were drilled or sub-drilled and reamed in the laboratory. The rivets were driven in a large commercial shop in accordance with good practice. The high tensile bolts were tightened in the laboratory to reach the minimum tension of 25,600 pounds specified for the 3/4 inch diameter bolt.

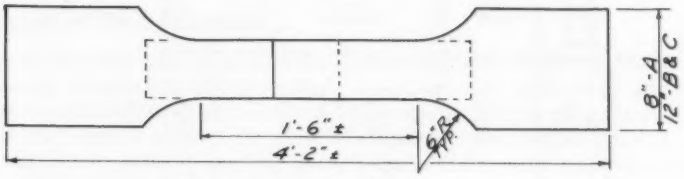
Static Test Results

Some of the rivets were removed after driving, and tested for shear strength. Based on driven rivet areas, the carbon steel rivets were found to

Material	Kind of Steel	Yield Strength for 0.2% offset (lb.in.$^{-2}$)	Ultimate Strength (lb.in.$^{-2}$)	% Elongation		% Reduction in Area	Hardness, Rockwell "B"
				8" Gage Length	2" Gage Length		
A	High-Strength	51,600	77,800	26.4		62.1	83.1
G	Low-Alloy (ASTM A242-50T)	53,600	75,200	26.7		60.4	80.4
I	Carbon (ASTM-A7)	38,600	70,250	26.5		54.2	80
K	" "	34,100	57,600	33.6		63.0	
L	Silicon (ASTM-A94)	45,800	81,900	20.0		56.2	87
M	" "	56,600	92,900	20.9		45.9	88.2
N	" "	48,800	84,500	20.2		54.1	87
P*	High Strength Steel	56,450	89,250		24.0	59.6	
Q*	Weldable Structural Steel (ASTM-A373)	37,500	60,720	31.5			
T	High Strength Steel	66,600	89,400	23.2	47.5	63.3	86.4

* Data Furnished by Steel Company

TABLE 1. PHYSICAL PROPERTIES OF THE MAIN PLATE MATERIALS

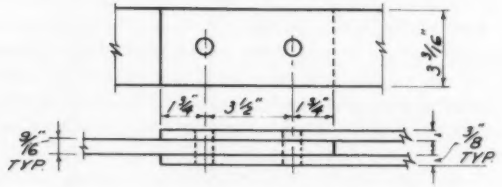


TYPICAL PLAN
ALL PATTERNS

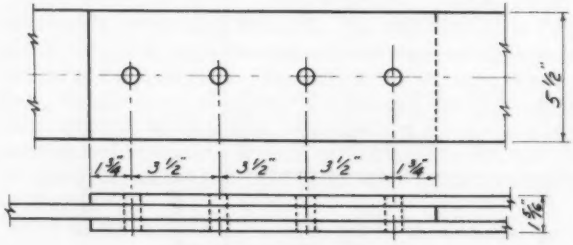
ALL FASTENERS:
3/4" NOM. DIA.

ALL HOLES:
13/16" DRILL

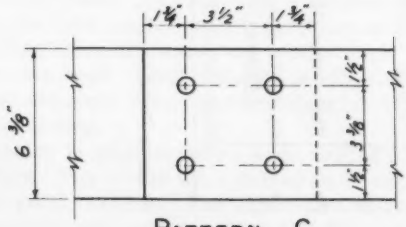
STRESS RATIOS:
T : S : B =
1.00 : 0.75 : 1.58



PATTERN A



PATTERN B



PATTERN C

AREAS:	PATTERN A	B	C
TENSION	1.34	2.64	2.67
SHEAR	1.77	3.53	3.53
BEARING	0.84	1.69	1.69

SPECIMENS FOR TESTS OF JOINTS, STATIC AND FATIGUE

FIG. 1

have an ultimate shear strength of 47,000 psi and the manganese steel rivets 54,500 psi, or 15 per cent higher. A study of clamping force in 128 of the carbon steel rivets was made by Mr. Y. Yoshimi, using the relaxation method. He found that after the fatigue failure of the joints, the clamping force of the individual rivets in them varied from 1,000 to nearly 20,000 pounds. Thirty per cent had less than 5,000 pounds; 20 per cent had over 11,000 pounds; the other half were in between.

The results of 26 static load, tension tests of riveted joints of seven steels are presented in Table 2. In many cases, initial failure occurred by double shear of the rivets, with the manganese steel rivets showing definitely higher strength, approaching and in six cases exceeding the strength of the main plate. Upon failure of the rivets, they were removed, replaced by high strength bolts, and the specimen retested. (It had been shown in other research that this procedure had little effect on main-plate strength.) So in all cases the main plate, which in fatigue proved critical, was tested to failure. The results showed correlation, within a few per cent, of actual plate strength with strength predicted from the coupon properties of the main plate materials and the geometry of the joints. Three different rules for predicting joint efficiencies were applied, and the degree of correlation varies somewhat for them, but all three predict higher efficiency (5 to 12 per cent) for Pattern B joints than for Patterns A or C. In fact, in Pattern B, the joints developed 80 to 88 per cent efficiency, averaging 84.6 per cent; while in Patterns A and C the joints developed 76 to 82 per cent efficiency, averaging about 79 per cent. Patterns A and C differed only about 1 per cent in efficiency. In general, then, static tests showed plates in the riveted joints had ultimate static strengths closely related to the coupon properties of the main plate materials.

Fatigue Test Results

For reasons both of brevity and of clarity, not all the results of the fatigue test programs are presented in the present summary paper. From Program I, only those steels are considered which were subsequently tested in the form of riveted or bolted joints. From Program II all of the tests of joints of Pattern B have been omitted. These joints had a higher predicted and actual efficiency in static tests than did those of Patterns A and C; however, in fatigue tests no clear superiority was evident for this pattern. Direct comparison would be confusing; but the results would not alter the conclusions.

The fatigue test results are given in Tables 3 - 5 and are presented graphically in Figs. 2 - 7. The tabulated maximum tensile stress in each cycle was obtained by dividing the load applied by the testing machine, by the net section of the main plate material: in the figures, this value is plotted to linear scale as the ordinate. For the abscissa, which represents the number of stress cycles to failure, a non-linear scale has been selected. This scale is proportional to the square root of the number of cycles. Other scales, linear and non-linear, were considered. The choice was based not on any rational theory, but on visual clarity plus the implication that, with increasing life, numerical differences diminish in importance.

Of the 15 steels tested in the form of a plate with a hole, results are given in Table 3 and shown in Fig. 2 for those eight steels subsequently tested in joints. The plates were 5/8" thick; the test section of the plate, 4" wide by 16" long, had a hole reamed to 1-1/16" diameter in its center. By comparison with the data in Table 1, it can be seen that there is a fairly good correlation between the static and fatigue test results; the steels with higher

Steel	ASTM Spec.	Fastener Pattern	Fastener Type *	Edge Conditions	Ultimate Load for Failure: Rivets lbs.	Ultimate Load for Failure: Plate lbs.	Ultimate Stress for Failure: Rivets p.s.i. (c)	Ultimate Stress for Failure: Plate p.s.i. (d)	Type of Plate Fracture	Plate Efficiency %
I	A7	A	C	(a)		95,000		71,000	Brittle	75.5
L	A94	A	C	(a)	96,000	118,000	54,300	88,900	Ductile	79.2
L	A94	A	C	(a)	98,500	116,000	55,700	86,900	Brittle	77.9
N	A94	A	C	(b)	103,000	115,400	58,300	86,500	Brittle	77.5
N	A94	A	C	(b)	100,500	121,000	56,900	90,600	Brittle	81.3
L	A94	A	M	(a)	110,000	119,400	62,200	89,500	Ductile	80.2
L	A94	A	M	(a)	111,000	116,000	62,800	86,900	Ductile	77.9
I	A7	B	C	(a)		180,000		68,200	Ductile	83.0
I	A7	B	C	(a)		174,600		66,100	Brittle	80.5
L	A94	B	C	(a)	190,000	223,000	53,800	84,500	Ductile	86.8
L	A94	B	C	(a)	186,000	211,000	52,700	80,000	Ductile	82.1
N	A94	B	C	(b)	188,000	226,000	53,300	85,600	Ductile	88.0
N	A94	B	C	(b)	193,000	218,000	54,700	82,600	Ductile	84.8
L	A94	B	M	(a)		225,000		85,300	Ductile	87.5
L	A94	B	M	(a)		219,000		83,000	Ductile	85.2
I	A7	C	C	(a)	189,000	198,000	53,500	74,100	Ductile	78.6
L	A94	C	C	(a)	186,000	238,000	52,700	89,100	Ductile	79.9
L	A94	C	C	(a)	184,000	228,000	52,100	85,500	Ductile	76.5
N	A94	C	C	(b)	190,000	230,000	53,800	86,100	Ductile	77.2
N	A94	C	C	(b)	188,000	231,000	53,300	86,600	Ductile	77.5
L	A94	C	M	(a)	230,000	241,000	65,100	90,400	Ductile	80.8
L	A94	C	M	(a)	229,000	236,000	64,900	88,500	Ductile	79.2
K	A7	C	M	(a)		169,000		63,300	Ductile	82.0
G	A242	C	M	(a)		221,000		82,800	Ductile	82.1
A	A242	C	M	(a)		225,000		84,300	Ductile	81.0
M	A94	C	M	(a)		266,000		99,600	Ductile	80.1

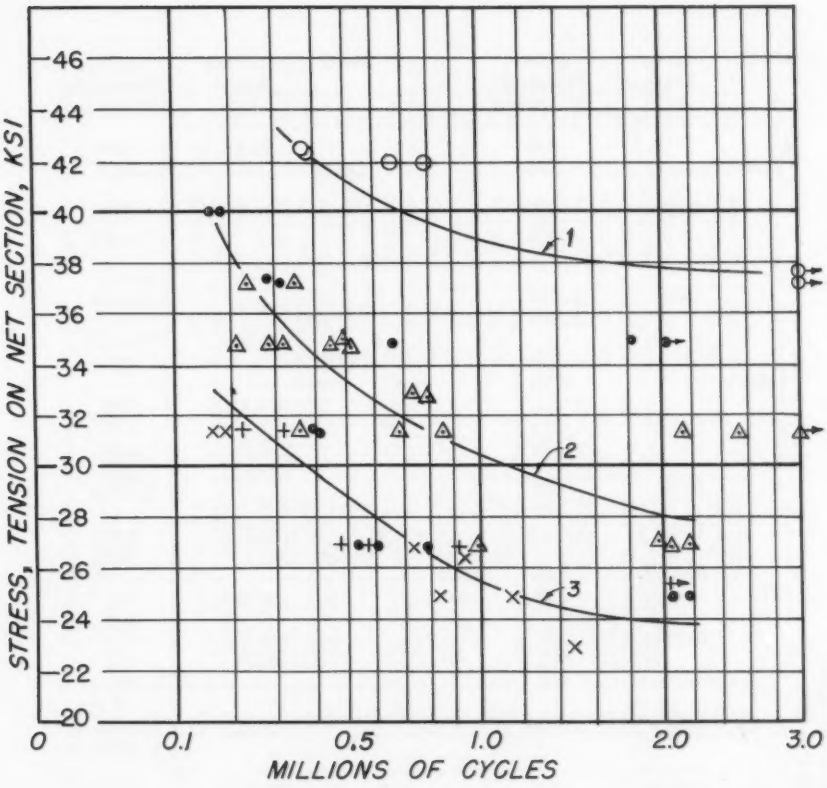
(a) = Milled
(b) = Machine flame cut and post-heated
(c) = Based on nominal diameter of rivets
(d) = Based on net section of main-plate
* C = Carbon steel M = Manganese steel

TABLE 2. RESULTS OF THE STATIC TENSION TESTS OF THE RIVETED JOINTS

Steel	Stress Range ksi	Life, Thousands of Cycles
A (A242)	0-31.5	398
		398
	0-27	530
		601
		793
	0-25	2,030
		2,080
G (A242)	0-40	159
		192
	0-37.5	293
		304
	0-35	657
		1,803
	0-31.5	2,013*
I (A7)	0-31.5	216
		312
	0-27	492
		561
		901
	0-25	2,000*
K (A7)	0-31.5	167
		190
	0-30	382
	0-27	730
		935
	0-25	835
		1,156
	0-23	1,428
L (A94)	0-35	209
		446
	0-31.5	852
		2,509*
	0-27	1,916
		2,012*
M (A94)	0-35	281
		318
	0-31.5	368
		688
	0-30	437
	0-27	981
		2,137*
N (A94)	0-37.5	234
		341
	0-35	493
		503
	0-33	733
		775
	0-31.5	2,137*
		3,160*
T	0-42.5	381
		386
	0-40	648
		785
	0-37.5	3,000*
		3,000*

* Indicates specimen did not fail

TABLE 3. FATIGUE TESTS, PLATES WITH A HOLE



(→) ARROW SHOWS NO FAILURE

KEY

STEEL	SPEC.
•	A, G	A242
+	I	A7
×	K	A7
△	L, M, N	A94
O	T	AREA

FIG. 2. PLATES WITH A HOLE

physical properties had in general higher fatigue strength also. The seven steels not shown give similar results. One steel of those shown, Steel T, was originally made to a Navy specification (now it is commercially available); it is definitely higher in both static and fatigue properties, as shown on this figure, so that a separate curve has been drawn for it. The A7 steels are seen to fall well below the high strength steels, as their static properties would lead one to expect. The remaining five high strength steels are represented by a third curve, intermediate between the other two. At one million cycles, the difference in fatigue strength between the A7 steels and the five high strength steels is about 5 ksi.

A direct comparison of manganese steel rivets with carbon steel rivets, with no other variable than the fastener, was made in the case of one steel only. The results, shown on Fig. 4, do not show that either rivet produced a joint clearly stronger than the other under the conditions of the tests. If there is any advantage, it appears to be with the manganese steel rivet. Large scatter in results occurred in the groups of specimens not presented, as well as those shown. The short grip, 1-5/16", may have affected the present comparison of rivet steels; at longer grips, the manganese steel rivets might more uniformly develop a higher clamping force. Results using these rivets have been excluded from the remaining tables and figures.

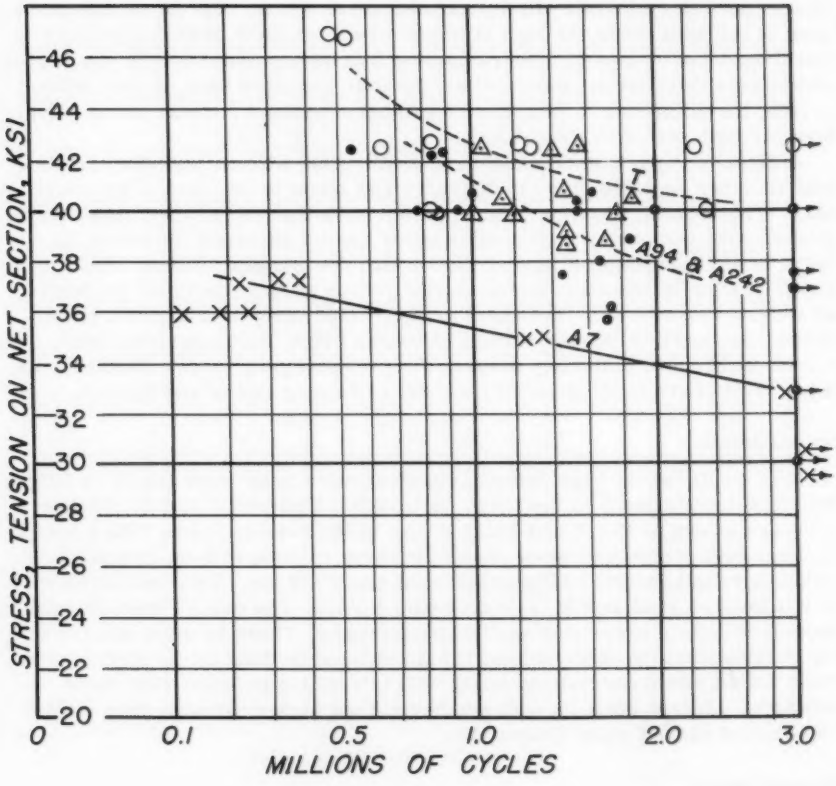
Bolted Joints

Five steels (of the eight tested as plates with a hole) were tested in fatigue, with high tensile bolts as fasteners, in joints of Patterns A and C. The results are shown in Fig. 3 and Table 4. As in the tests of plates with a hole, the very high yield-point steel shows above the others, with an indicated fatigue strength, at one million cycles, of nearly 43 ksi. This compares with 39 ksi for this steel in Fig. 2, Plates with a Hole. The one A7 steel (Steel K) showed definitely lower average fatigue strength. There is more scatter in the results from the other steels; but it can be seen that fatigue strengths in these bolted joints correspond fairly well in relative position with static strengths. Silicon steel M, with lower yield but higher ultimate than steel "T" showed almost equal fatigue life.

Riveted Joints

At this point, approach is made to the heart of our original problem: that of comparing the strength, in fatigue, in riveted joints, of various steels. From the results shown on Fig. 5 it can be seen that, as in Fig. 4, the scatter which characterized all the tests of riveted joints is evident. All specimens of Fig. 5 were Pattern C. In the outlined area appear the results of an effort to reduce scatter effects by testing more specimens at a single stress: except for the different plate materials, all the specimens were as closely alike as possible. All these tests were at zero to 30 ksi. The contrast between the near equality of the fatigue lives of these six steels, with the great differences in their static properties, may be observed.

One steel, furnished to specification A373 (structural steel for welding), a late addition to the program, seemed to show exceptionally long fatigue life under the same test conditions. Two of the specimens ran beyond 3 million cycles. The ductility of this steel was 31.5 per cent, not higher than steel K. No explanation of the result from these few tests on this steel is offered. Later tests under somewhat different conditions failed to support the superiority indicated here.



KEY

STEEL	SPEC.
● ---- A, G ----	A242
X ----- K ---	A7
△ --- L, M, N ---	A94
○ ----- T ----	AREA

(→) ARROW SHOWS NO FAILURE

FIG. 3. FATIGUE TESTS OF BOLTED JOINTS

Steel/Pattern	Stress Range ksi	Life, Thousands of Cycles
A/A	0-40	1,552
		948
	0-38	1,671
	0-37	3,000*
	0-33	3,000*
A/C	0-40.5	1,507
		985
	0-39	1,825
	0-37.5	3,000*
		1,452
G/A	0-40	3,000*
		2,054
		942
	0-36	1,737
		1,720
	0-30	3,000*
G/C	0-42.5	889
		821
		542
	0-40.5	1,627
		1,609
		977
	0-30	3,000*
	0-27	3,000*
K/A	0-36	237
		195
		112
	0-33	3,000*
		2,905
	0-30	3,000*
K/C	0-37.5	382
		312
		225
	0-35	1,337
		1,244
	0-30	3,000*
M/A	0-42.5	1,567
		1,410
		1,057
	0-40	1,760
		1,205
		1,008
M/C	0-40.5	1,810
		1,473
		1,145
	0-39	1,695
		1,490
		1,459
T/A	0-42.5	3,000*
		1,283
		614
	0-40	2,343
		899
		874
T/C	0-47	522
		499
	0-42.5	2,254
		1,254
		822
	0-40.5	2,398x
		1,141x

* Specimen did not fail

x Failed in Grip

TABLE 4. FATIGUE TESTS OF BOLTED JOINTS

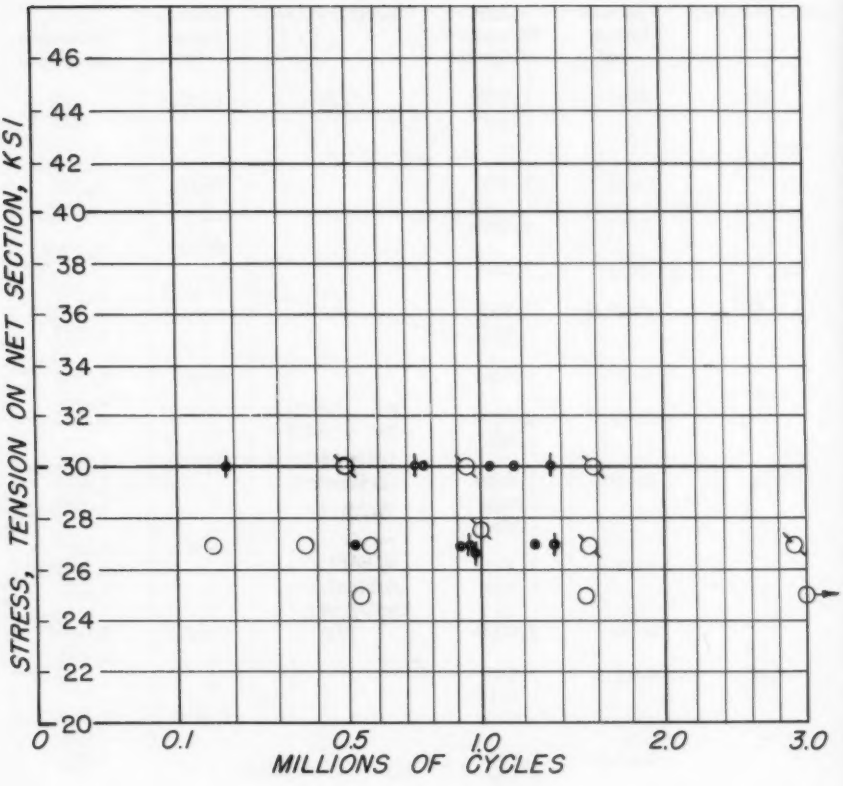
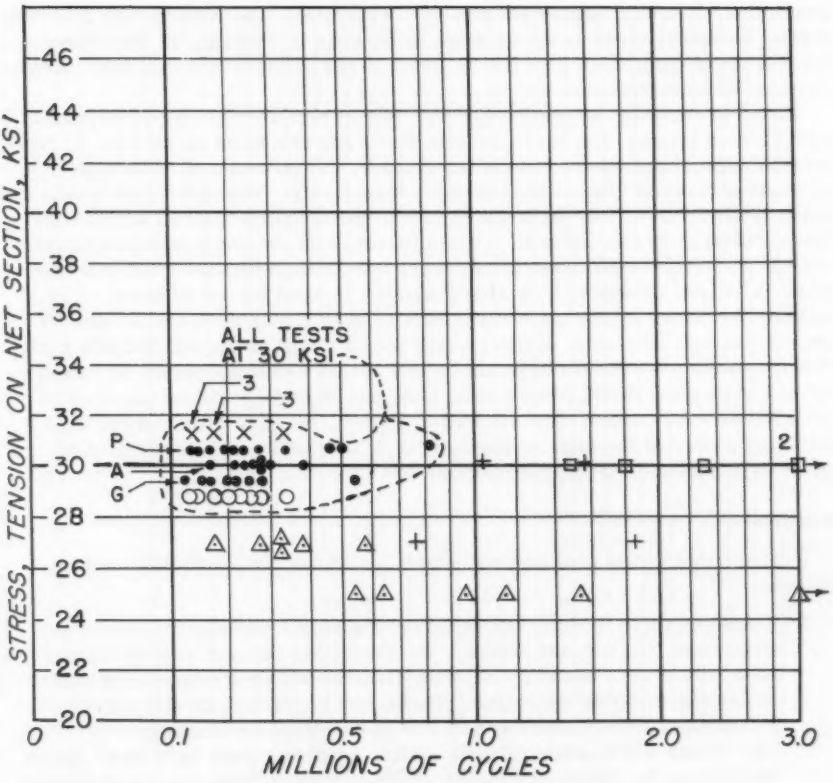


FIG. 4. FATIGUE TESTS OF RIVETED JOINTS
CARBON VS. MANGANESE STEEL RIVETS



(→) ARROW SHOWS NO FAILURE

ALL RIVETS A141
ALL JOINTS PATTERN "C"

KEY

STEEL	SPEC.
•---A,G,P---	A242
+---- I ---	A7
X---- K---	A7
△---L,N---	A94
□-----Q---	A373
O-----T---	AREA

FIG. 5. FATIGUE TESTS OF RIVETED JOINTS

On Fig. 6, 16 points for specimens of Pattern A are superimposed on the previous chart. The new points are shown by large symbols. The cluster of tests at zero to 30 ksi is represented by the shaded area. In spite of the fact that Pattern A, with two rivets, has the same parameters as Pattern C it appears that somewhat higher fatigue life is obtained. As between the different steels, however, there is no apparent difference in results. In this respect, the additional points confirm the absence of correlation between static properties and fatigue strength.

Fig. 7 presents a comparison of the fatigue test results of bolted joints with riveted joints. The three curves above are the same as on Fig. 3, representing approximately the trends for Steel T, the silicon and A242 high strength steels, and the carbon steels respectively. The individual points are not replotted, but all the failures of bolted specimens occurred within the dotted outline. In the cases of riveted joints (with the one possible exception, the A373 steel), the different steels were not distinguishable from one another in fatigue strength; so a single symbol is used for all of them. The highest maximum cyclic stress at which riveted joints were tested was 30 ksi, at which stress most of the riveted joints failed at around 200,000 cycles. The few specimens of bolted joints tested at this same maximum stress did not fail even after 3,000,000 cycles. Only one bolted specimen failed at 33 ksi; this was the carbon steel of lowest yield point. The effect of the high clamping power of the high strength bolts in increasing the resistance of joints to repeated load is apparent.

Discussion

Before considering conclusions, some qualifying considerations are as follows:

1. In all the tests to date, the stress cycle employed was from zero to maximum tension; and further, the maximum tension was set rather high with a view toward producing failures within a reasonable period of testing machine operation. These test conditions do not correspond to very common service conditions in buildings or bridges.
2. The rivets were comparatively short. Longer rivets have been shown to develop, by virtue of greater shrinkage on cooling, higher clamping power; hence the comparison made herein of carbon and manganese steel rivets and high tensile bolts may not be equally applicable to joints of greater thickness.
3. There was a wide variation observed in the surface condition of the plates of the various steels. Some were quite smooth; others rough or pitted in varying degree. Examination of the results does not show the roughest plates to be the weakest in fatigue, or vice versa; but there was some indication of such an effect. Since all specimens of any one steel were taken from a single plate, this factor may have influenced the results.
4. The symmetry of clamping forces, not only the average clamping force, of the fasteners within a joint, probably influenced the fatigue life. This factor of possible or probable asymmetry of clamping force with resulting variations in stress concentration, may well be of dominant importance in at least some of the results.

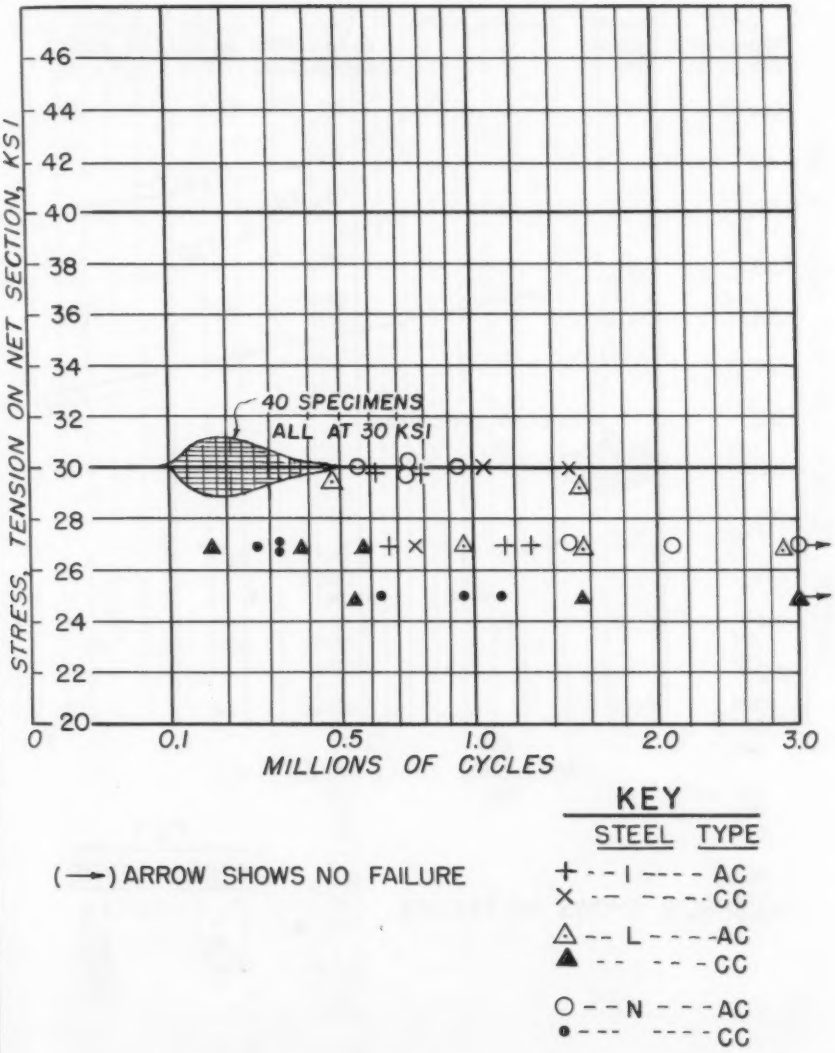


FIG. 6. FATIGUE TESTS OF RIVETED JOINTS

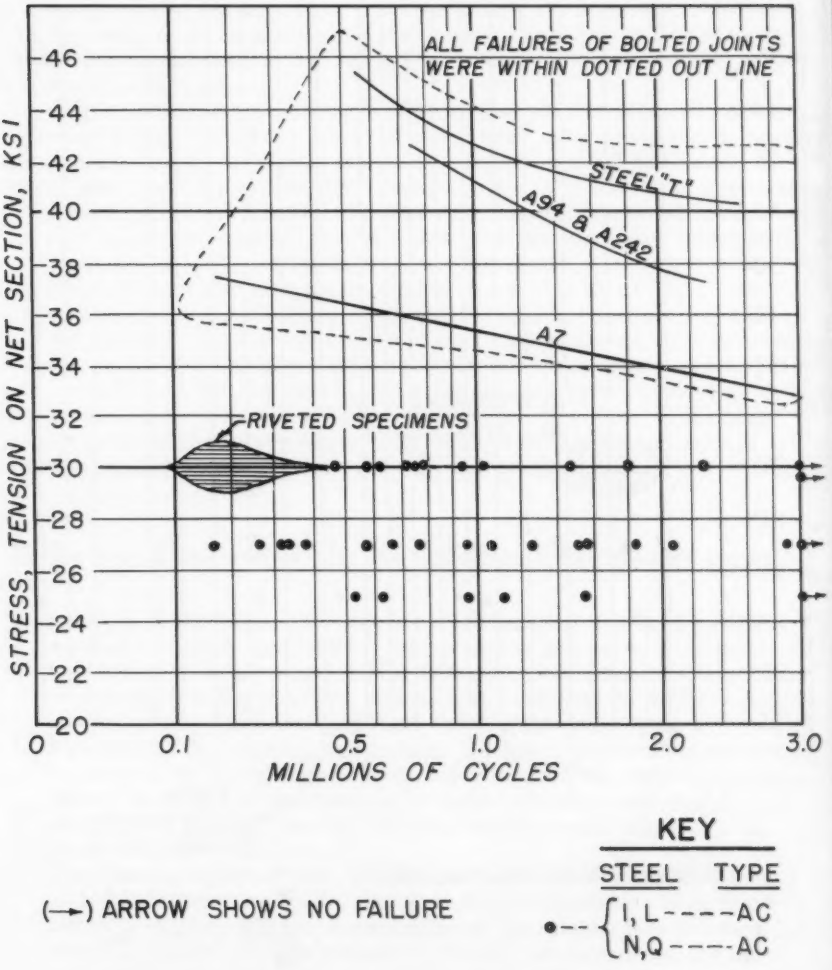


FIG. 7. FATIGUE TESTS OF RIVETED JOINTS COMPARED TO BOLTED JOINTS

Steel/Pattern	Stress Range ksi	Life, Thousands of Cycles
A/C	0-30	368
		294
		275
		253
		230
		221
		218
		161
G/C	0-30	572
		279
		241
		209
		200
		166
		160
		110
I/A	0-30	775
		612
	0-27	1,282
		1,082
		681
I/C	0-30	1,557
		1,015
	0-27	1,820
		781
K/C	0-30	319
		232
		182
		181
		179
		153
		153
		149
L/A (M)	0-30	1,320
		750
		194
	0-27	1,346
		980
		966
L/C (M)	0-30	1,151
		1,022
		767
	0-27	1,255
		914
		528
L/A	0-30	1,584
		922
		496
	0-27	2,924
		1,579
		992
L/C	0-27	576
		384
		168
	0-25	3,000*
		1,570
		535
M/A	0-30	760
		754
		579
	0-27	3,000*
		2,090
		1,524
M/C	0-27	311
		309
		267
	0-25	1,120
		966
		621

(M) Manganese steel rivets; all other joints fastened with carbon steel rivets.

* Specimen did not fail

TABLE 5. FATIGUE TESTS OF RIVETED JOINTS

Steel/Pattern	Stress Range ksi	Life, Thousands of Cycles
P/C	0-30	814
		501
		483
		323
		227
		211
		211
		185
		174
		173
Q/C	0-30	5,000*
		3,000*
		2,300
		1,790
		1,490
T/C	0-30	336
		257
		216
		204
		200
		173
		149
		147

* Specimen did not fail

TABLE 5. FATIGUE TESTS OF RIVETED JOINTS (CONT'D)

CONCLUSIONS

From the test results and discussion, the following conclusions are drawn:

1. In the bolted joints, the fatigue strength is related to the static-test properties of the plate material.
2. Joints fastened with rivets of manganese steel, A195, were not clearly superior to similar joints using ordinary carbon steel rivets, A141, in fatigue strength.
3. As tested in fatigue in riveted joints, no plate steel showed superiority over A7 steel to correlate with higher static strength. One possible exception, the A373 steel needs further testing before a conclusion relating to it can be stated.
4. The joints connected with high strength bolts show much higher fatigue strength than those connected with rivets, even approaching the yield points of some of the steels.
5. The most important variable governing the fatigue strength of a joint is clamping force. In riveted joints, as tested, the variations—and the distribution within a joint of those variations—of clamping force, subordinates whatever superiority one steel has over another. Joints having uniformly high clamping force, such as the bolted joints, have high fatigue strength.

The project committee proposes, in future programs, to pursue further the evaluation of high-strength steels in fatigue, by more careful isolation or measurement of the variables and by increasing the number of like specimens tested. In view of the importance the committee assigns to the factor of stress concentration at the edges of the hole due to bearing, tests have been initiated to establish a "lower limit" under a control condition of full bearing, no clamping. Tests using less severe stress cycles than zero to maximum tension—such as 1/2 S to S—which occur more commonly in bridges—are also planned.

ACKNOWLEDGMENT

This paper has been prepared on behalf of the engineers who performed the research and also on behalf of Project Committee VI, of which the author is presently chairman, of the Research Council on Riveted and Bolted Structural Joints. The Council sponsored, and the Committee in general guided the research, which was carried out at Northwestern University during the years 1952-1957. The responsible research men include: Frank Baron, B. J. Covington, K. J. Kenworthy, E. W. Larson, L. B. McCammon, C. W. Muhlenbruch, T. M. Ridley, L. T. Wyly, and Y. Yoshimi. The author wishes to express his appreciation for suggestions and criticism during the preparation of the paper to members of the Project Committee and to the research men who helped by consultation or correspondence.

Journal of the
STRUCTURAL DIVISION
Proceedings of the American Society of Civil Engineers

EFFECTS OF FABRICATION TECHNIQUES ON BOLTED JOINTS

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SYNOPSIS

A considerable amount of earlier investigation of the characteristics of structural joints assembled with high tensile bolts(1) were focused upon the possibility of slip in the range of currently used design stresses. Other tests clarified the behavior of such joints under fatigue loads.

Some questions about the possible effects of various fabricating processes still were open, some others arose as new techniques and ideas developed both in methods of tightening the fasteners and in design principles. The question of the efficiency of joints being influenced by such factors as punching of the holes, misalignments and the paint on the faying surfaces has been left for the present investigations.

The high practical value found in the one-turn-of-the-nut method drew the attention to its possible effects upon the behavior of joints. This was especially so in the case of large and long joints, in which the gradual development of slip required more insight into the behavior of such joints.

A certain tendency toward considering the coefficient of friction in the design of the joint made a more thorough study of this factor necessary. Thus the threefold scope of the later investigations conducted on high tensile bolted joints.

Tests on joints with drilled and with misaligned punched holes, both with mill scale and with painted faying surfaces showed that, as expected, the punching of the holes does and the misalignment and the paint do not affect significantly the efficiency of the joint. The effect of punching of the holes proved to be the same for bolted joints as for riveted ones. As a matter of

Note: Discussion open until August 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1973 is part of the copyrighted Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. ST 3, March, 1959.

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fact, the efficiency of the bolted joint in view of these effects, can be sufficiently predicted by formulae used to estimate the efficiency of riveted joints. The condition of the faying surfaces, whether painted or unpainted, apparently does not affect the efficiency.

A sufficient number of small (4 bolt) joint tests and a series of tests on large joints proved that the one-turn-of-the-nut method acts the way it would be predicted from the presence of the higher clamping force.

Systematic analysis of all available data on slip tests throws more light upon the behavior of the coefficient of friction in the joints. It shows that the major factor determining its value is the treatment of the faying surfaces. Such factors as clamping force, tension:shear ratio, etc., have well defined but only minor influence on the value of the coefficient of friction.

These results and others should be of great value in the drafting of specifications for joints assembled with high tensile bolts.

I. The Effect of Punched Holes, of Misalignment, and of Painted Faying Surfaces

Fifteen tests of double-lap joints assembled with 12 high tensile bolts of 3/4-in. diameter and four tests of double-lap joints with only one row of 4 high tensile bolts of 7/8-in. diameter have been run. Some of the tests were conducted at -24 F, others at room (between 66 and 76 F) temperature. The slip and the elongation of the joints were measured. The first major slip load, ultimate strength, energy absorption and efficiency of the joints were studied.(2)

A. Specimens

1. Description

Fifteen 12-bolt specimens, designated Type U and Type X (Fig. 1), were tested. They were made of 3/4-in. and 3/8-in. plates of ASTM A 7-51T semi-killed steel, and joined by twelve 3/4-in. high tensile steel bolts (ASTM A 325) in 13/16-in. diameter drilled or punched holes. The holes were misaligned as shown in Fig. 1. All bolts were tightened to a torque of 320 foot-pounds, the recommended approximate equivalent torque for required minimum tension(3) in 3/4-in. bolts. This torque corresponds to an average tension in the bolts of 26,000 pounds.

Two types of specimens were tested: Type U with a tension:shear ratio of 1.00:0.75 and Type X with a tension:shear ratio of 1.00:1.00. Specimens of each type were tested under the following conditions: (1) drilled holes, mill-scale faying surfaces, room temperature; (2) drilled holes, red lead faying surfaces, room temperature; (3) punched holes, red-lead faying surfaces, room temperature; (4) punched holes, red-lead faying surfaces, minus twenty-four degrees Fahrenheit. Two specimens of each type were tested except for condition (2).

To this series of 12-bolt specimens four 4-bolt specimens made of 1/2-in. and 1/4-in. plates of ASTM A 7-51T semi-killed steel have been added, joined by 7/8-in. high tensile bolts in 15/16-in. drilled holes in one row. These are also shown on Fig. 1 and designated Type ZU and ZX, the first having a tension:shear ratio of 1:0.75, the second one of 1:1. Two of each type were tested, both with painted faying surfaces, and one in each set of two having one outside hole misaligned as shown on Fig. 1.

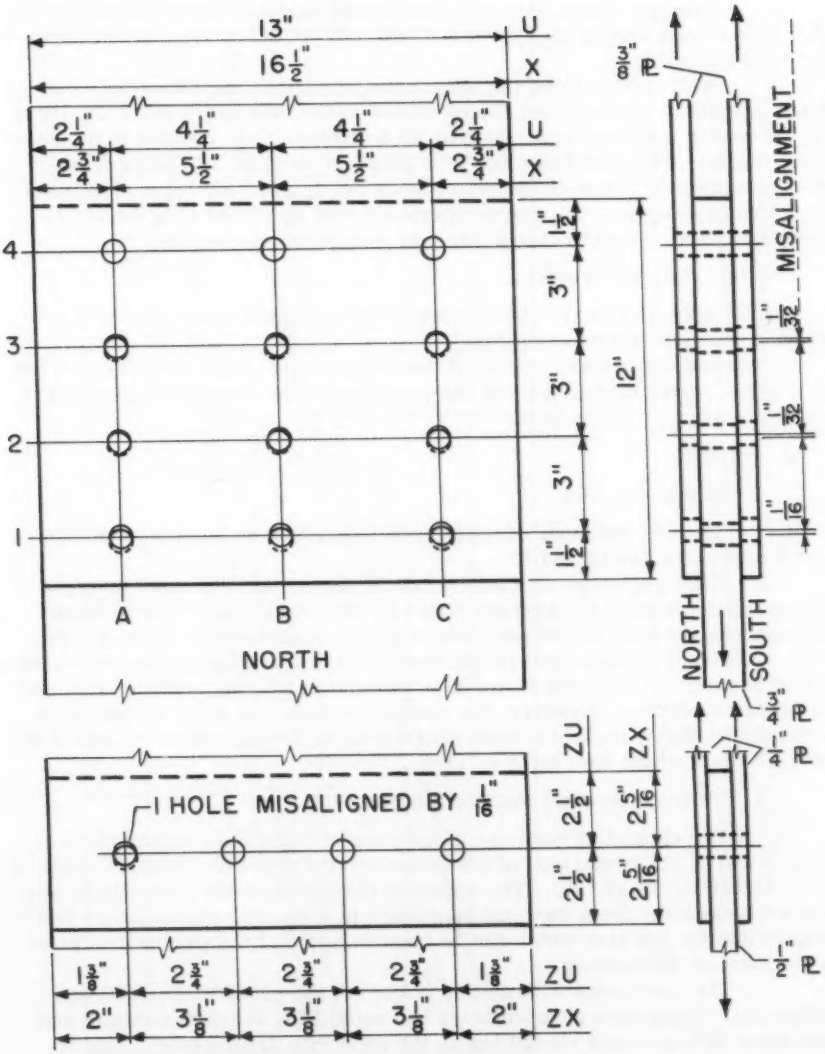

Fig. 1 Specimen Types U, X (top) and ZU, ZX (bottom).
U and ZU tension:shear ratio 1:0.75
X and ZX tension:shear ratio 1:1

2. Preparation

The specimens were cut, shaped, and drilled in the structural shop of the Laboratory, the punching was done in the Seattle plant of the Bethlehem Pacific Coast Steel Corporation.

The specimens with red-lead faying surfaces were painted (on the faying surfaces only) with Type 1 red lead (TT-P-86) at least seven days prior to assembly.

While assembling the specimens, the tightening of the bolts has been gradual in six separate stages (three on the laboratory floor and three in the testing machine) assuring that all bolts have been brought to the same final torque. After the bolts had been properly torqued, the slip gages, thermocouples (for low-temperature tests only), and slide-wire elongation gages were mounted. If a low-temperature test was to be run, the entire specimen from clevis to clevis was enclosed in the cooling bag.

B. Description of Tests

All specimens have been loaded in the 2,400,000-pound Baldwin hydraulic testing machine to failure.

Measurements were taken of the temperature of the specimen, of the slip at the center of each side of the joint, and of the elongation along three lines on each face of the joint.

C. Results

1.. Slip in the Joint

The slip behavior of specimens U and X is summarized in Figs. 2 and 3, and in Tables IA and IB.

The specimens with mill-scale faying surfaces showed no signs of appreciable slip until the average shear on the gross area of the bolts had become greater than 15,000 psi. For a design tension:shear ratio of 1.00: 0.75, 15,000 psi is equivalent to the working load. The specimens which were tested at minus 24 degrees Fahrenheit with painted faying surfaces behaved in the same manner. However, the specimens tested at room temperature with painted faying surfaces began slipping at an average shear of only 5,000 psi (1/3 the working load for a ratio of 1,00:0.75).

2. Ultimate Strength and Elongation of the Joint

The elongation behavior of specimens U and X is summarized in Figs. 4 and 5. A comparison of the computed and observed ultimate loads is given in Tables IIA and IIB. The apparent elongation of the joint due to slip was not subtracted from the total elongation in preparing these curves and is responsible for the kink which can be seen on some of the curves just prior to the inset of the yielding.

The specimens with drilled holes had the greatest strength and elongation. Those with punched holes had about 90% as much strength and only about 60% as much elongation as the ones with drilled holes. Those which were tested at minus 24 degrees Fahrenheit had about 75% as much strength and only about 20% as much elongation as the drilled specimens tested at room temperature.

3. Type of Fracture

A short description of the plate fracture, including the per cent of cleavage present, is given in Tables IA and IB.

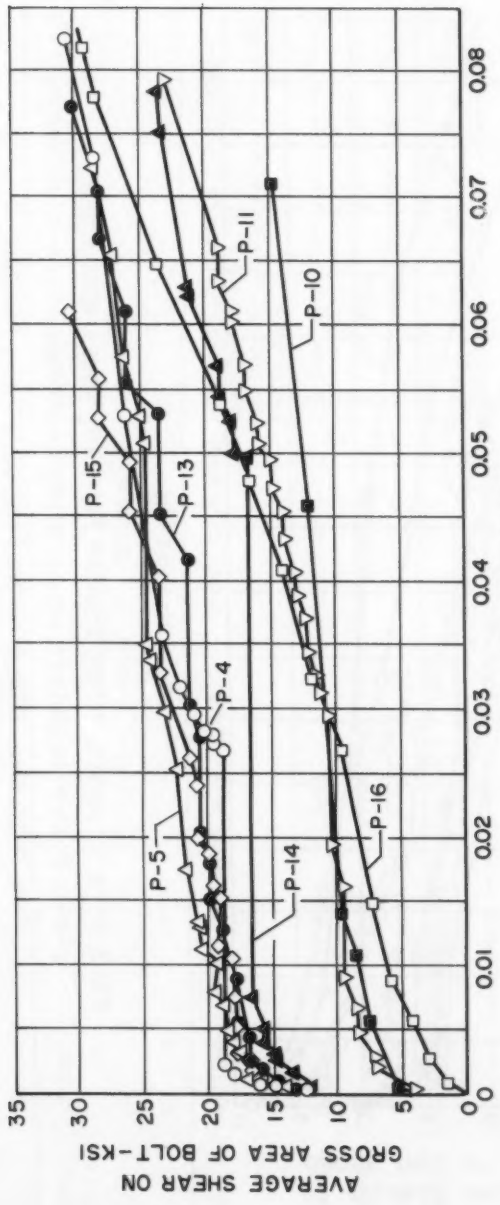

FIG 2. TYPE "U" JOINTS, TENSION: SHEAR RATIO 1:0.75. HOLES MISALIGNED.
P-4, P-5, P-16 DRILLED HOLES, OTHERS PUNCHED.
P-4, P-5, MILL SCALE FAYING SURFACE, OTHERS PAINTED.
P-13, P-14, P-15 TESTED AT −24 F, OTHERS AT ROOM TEMP.

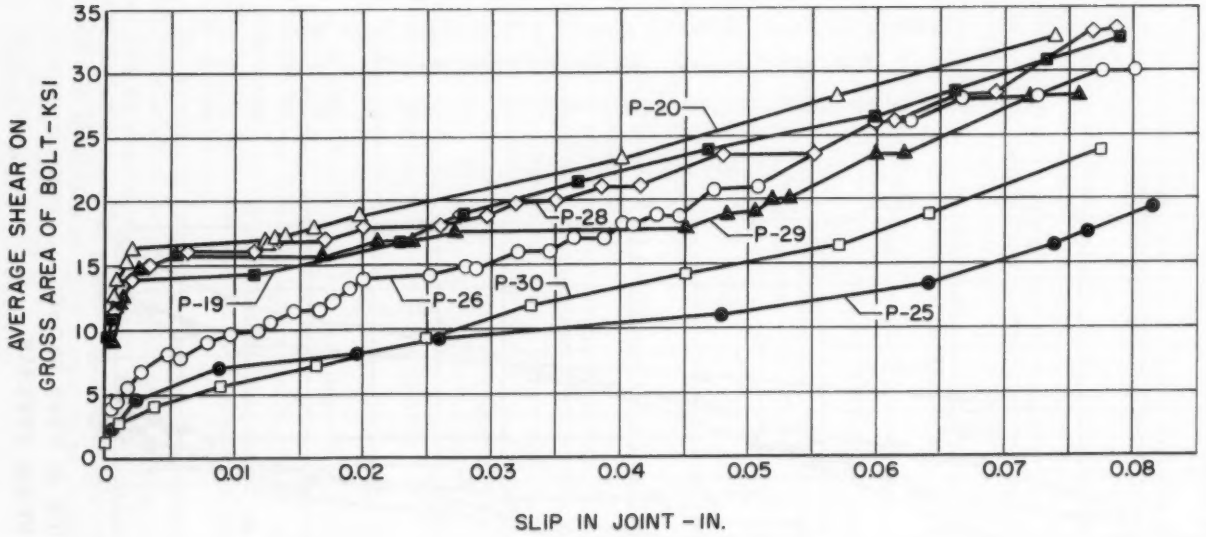

FIG 3. TYPE "X" JOINTS, TENSION: SHEAR RATIO 1:1. HOLES MISALIGNED.
P-19, P-20, P-30 DRILLED HOLES, OTHERS PUNCHED.
P-19, P-20, MILL SCALE FAYING SURFACE, OTHERS PAINTED.
P-28, P-29 TESTED AT -24 F, OTHERS AT ROOM TEMP.

TABLE I/A

DESCRIPTION OF PLATE FAILURE
TYPE U SPECIMENS
TENSION:SHEAR RATIO 1.00:0.75

Spec. No.	Holes	Faying Surface	Test Temp.	Avg. Shear at First Major Slip ksi	Avg. Shear at Ultimate Load ksi	Short Description of the Fracture, % Cleavage in the Fracture.
P-4	Drilled	Mill Scale	Room	18	47.7	100% shear, failure in 3/4-in. plate through first row of bolt holes. Considerable plastic deformation prior to fracture.
P-5	Drilled	Mill Scale	Room	18	47.9	17% cleavage. Otherwise same as P-4.
P-10	Punched	Red Lead	Room	6	41.6	80% cleavage. Otherwise same as P-4.
P-11	Punched	Red Lead	Room	5	42.8	10% cleavage. Otherwise same as P-4.
P-13	Punched	Red Lead	-24° F	16	46.0	100% cleavage, failure in 3/4-in. plate through first row of bolt holes. Some plastic deformation prior to fracture.
P-14	Punched	Red Lead	-25° F	17	30.6	100% cleavage. Practically no plastic deformation prior to fracture. Otherwise same as P-13.
P-15	Punched	Red Lead	-29° F	17	32.0	Same as P-14.
P-16	Drilled	Red Lead	Room	5.64	47.85	Failure in 3/4-in. plate through first row of bolt holes. Some plastic deformation prior to failure.

TABLE I/B

DESCRIPTION OF PLATE FAILURE
TYPE X SPECIMENS
TENSION:SHEAR RATIO 1.00:1.00

Spec. No.	Holes	Faying Surface	Test Temp.	Avg. Shear at First Major Slip ksi	Avg. Shear at Ultimate Load ksi	Short Description of the Fracture, % Cleavage in the Fracture.
P-19	Drilled	Mill Scale	Room	14	62.5	Failure occurred by shearing of all bolts. No plate fracture. All holes were elongated and deformed considerably indicating that all bolts came into bearing for some time before failure.
P-20	Drilled	Mill Scale	Room	17	61.1	Same as P-19.
P-25	Punched	Red Lead	Room	5	56.0	45% cleavage failure in 3/4-in. plate through first row of bolt holes. Considerable plastic deformation prior to fracture.
P-26	Punched	Red Lead	Room	5	55.6	6% cleavage. Otherwise same as P-25.
P-28	Punched	Red Lead	-24° F	15	40.0	100% cleavage failure in 3/4-in. plate through first row of bolt holes. Practically no plastic deformation prior to fracture.
P-29	Punched	Red Lead	-25° F	16	44.6	Same as P-28.
P-30	Drilled	Red Lead	Room	5.64	60.2	Same as P-19.

ST 3

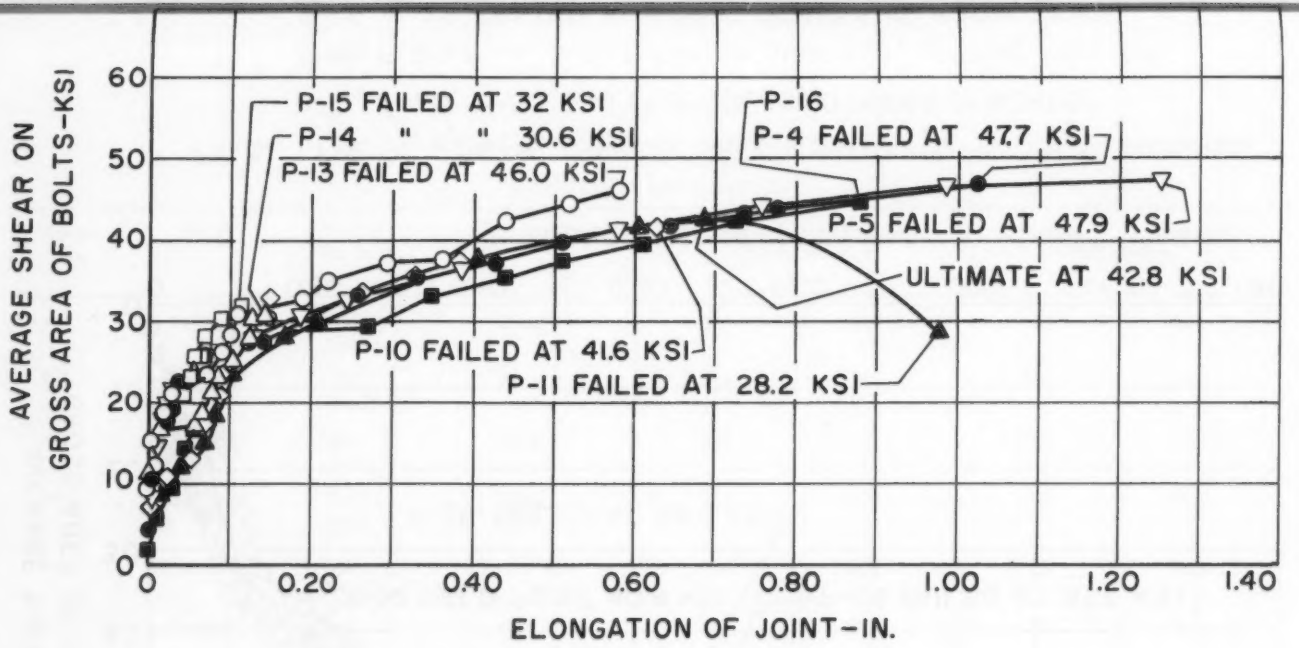

FIG 4. TYPE "U" JOINTS, TENSION: SHEAR RATIO 1:0.75. HOLES MISALIGNED.

P-4, P-5, P-16 DRILLED HOLES, OTHERS PUNCHED.

P-4, P-5, MILL SCALE FAYING SURFACE, OTHERS PAINTED.

P-13, P-14, P-15 TESTED AT -24 F, OTHERS AT ROOM TEMP.

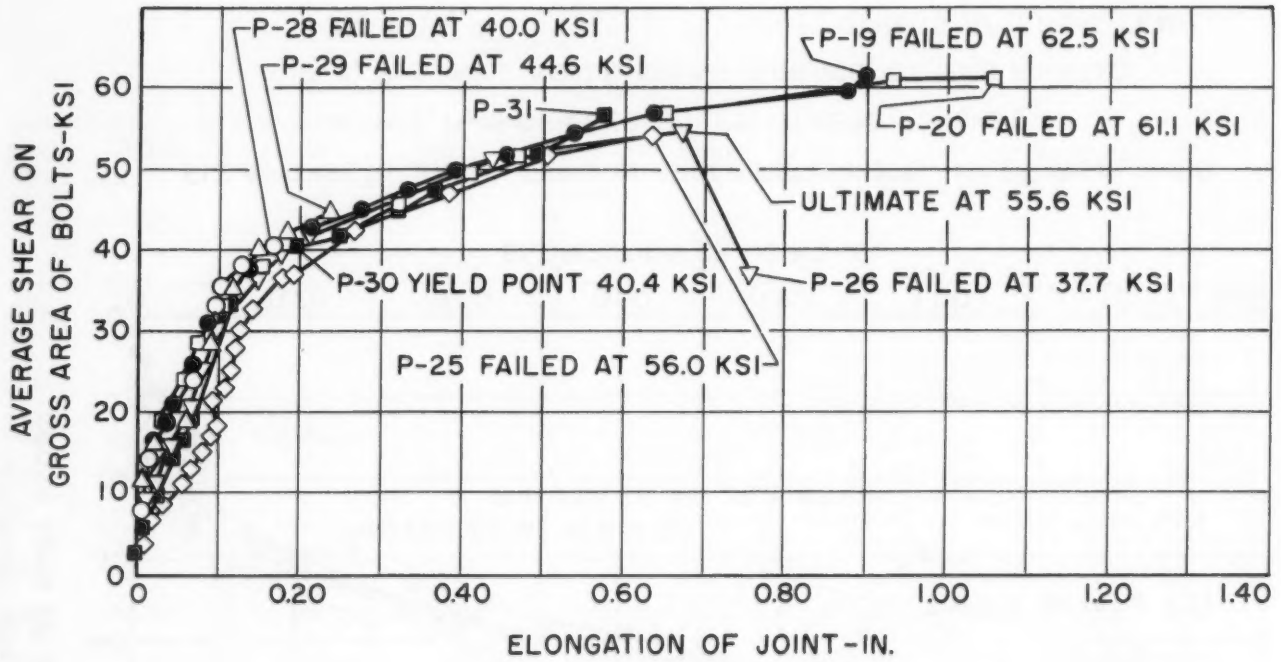

FIG 5. TYPE "X" JOINTS, TENSION: SHEAR RATIO 1:1. HOLES MISALIGNED.
P-19, P-20, P-30 DRILLED HOLES, OTHERS PUNCHED.
P-19, P-20, MILL SCALE FAYING SURFACE, OTHERS PAINTED.
P-28, P-29 TESTED AT -34 F, OTHERS AT ROOM TEMP.

P-19, P-20, MILL SCALE FAYING SURFACE, OTHERS PAINTED.

TABLE II/A

COMPARISON OF COMPUTED AND OBSERVED EFFICIENCY
TYPE U SPECIMENS
TENSION:SHEAR RATIO 1.00:0.75

Spec. No.	Holes	Faying Surface	Temperature	Computed Ultimate Load k	Observed Ultimate Load k	Computed Efficiency per cent	Observed Efficiency per cent
P-4	Drilled	Mill Scale	Room	515	507	87	85.6
P-5	Drilled	Mill Scale	Room	515	509	87	86.0
P-10	Punched	Red Lead	Room	444	442	75	74.6
P-11	Punched	Red Lead	Room	444	455	75	76.9
P-13	Punched	Red Lead	-24° F	444	487	75	82.4
P-14	Punched	Red Lead	-25° F	444	325	75	55.0
P-15	Punched	Red Lead	-25° F	444	340	75	57.5
P-16	Drilled	Red Lead	Room	522	508	87	84.7

TABLE II/B

COMPARISON OF COMPUTED AND OBSERVED EFFICIENCY

TYPE X SPECIMENS

TENSION:SHEAR RATIO 1.00:1.00

Spec. No.	Holes	Faying Surface	Temperature	Computed Ultimate Load k	Observed Ultimate Load k	Computed Efficiency per cent	Observed Efficiency per cent
P-19	Drilled	Mill Scale	Room	653	664	87	88.4
P-20	Drilled	Mill Scale	Room	653	648	87	86.4
P-25	Punched	Red Lead	Room	562	595	75	79.4
P-26	Punched	Red Lead	Room	562	592	75	78.8
P-28	Punched	Red Lead	-24° F	562	425	75	56.6
P-29	Punched	Red Lead	-25° F	562	475	75	63.3
P-30	Drilled	Red Lead	Room	661	640	87	84.2

All specimens U and X tested at room temperature underwent considerable plastic deformation prior to failure. Except for specimen P-13, all specimens tested at minus 24 degrees Fahrenheit underwent very little plastic deformation.

Specimens P-19, P-20 and P-30 (tension:shear ratio 1.00:1.00, drilled holes) failed by shearing of the bolts; all other specimens failed by fracture in the 3/4-in. plate through the first row of holes. All the holes in specimens P-19, P-20 and P-30 were found to be considerably elongated and deformed. This indicates that all the bolts had come into bearing long before the specimen finally failed.

Except for specimen P-10, all fractures which occurred at room temperature were primarily shear fractures; all specimens tested at minus 24 degrees Fahrenheit failed with nearly 100% cleavage fracture. The type of fracture present in the broken specimen was identified by visual observation.

4. Joint Efficiency

To compute joint efficiency the Relative Gage Method developed by F. W. Schutz, Jr. and N. M. Newmark was used.(4)

The efficiencies of the joints tested, both computed and actual, are summarized in Tables IIA and IIB. Note the close agreement between the computed and actual efficiencies for the specimens tested at room temperature. The specimens tested at minus 24 degrees Fahrenheit had (except for specimen P-13) actual efficiencies of approximately 25% less than the computed efficiency. The results of actual coupon tests were used in determining the theoretical efficiency of the joint and the theoretical ultimate load the joint could carry at 100% efficiency.

5. The Effect of Misalignment

Specimens with 12 bolts of Type U and X did not indicate any effect which could be attributed to the misalignment of the holes.

A more severe condition of misalignment was built in specimens Q-1 and Q-3, of the Type ZU and ZX (Fig. 1). These were compared with identical specimens Q-2 and Q-4, which did not have misalignment. The observed ultimate loads and efficiencies were as follows:

Specimen	Tension:shear ratio	Misalignment	Ult. load k	Efficiency per cent
Q-1	1:0.75	As shown	225	71.8
Q-2	1:0.75	None	235	74.9
Q-3	1:1	As shown	275	72.0
Q-4	1:1	None	302	89.1

These results consistently show that the specimens with severe misalignment support a lower ultimate load and lose efficiency. This leaves the problem of just how much misalignment would not affect the efficiency open and the question needs further study.

II. The Effect of the One-Turn-of-the-Nut Bolt Tightening Method Upon Slip and Efficiency of Joints

The general effect of the one-turn-of-the-nut method in large joints has been studied in the following tests. Two specimens of 80 bolts have been tested for slip and efficiency. One of the latter was a rather long joint. Twenty-six four-bolt specimens were tested in addition to these.(6)

A. Test and Test Method

1. Slip Test of 80-bolt Joints 153-M and 155-N

a. Preparation of specimens

The properties of these two specimens are listed in Table IIIA. The specimens are shown on Figs. 6, 7, 8 and 9. The properties of the steel are listed in Table IIIB.

TABLE III/A

PROPERTIES OF SPECIMENS

Spec. No.	Surface Condition	Net Ten. Area	Nominal Double Shear Area	Tension: Shear Ratio	Faying Area Per Bolt	Approx. Bolt Tension
		Sq. in.	Sq. in.		Sq. in.	kips
153-M	Mill Scale	92	125.6	1.00:0.73	57.6	70
153-M	Mill Scale	98	125.6	1.00:0.78	36.3	72.5
153-X	Mill Scale	33.94	42.4	1.00:0.80	55.8	69
155-Y	Mill Scale	33.25	47.1	1.00:0.71	30.6	72.5

TABLE III/B

AVERAGE PROPERTIES OF THE STEEL

Spec. No.	Plate Thickness	Upper Yield	Lower Point	Ult. Strength	Elong. in 2 in.	Red. of Area
		ksi	ksi	ksi	%	%
153	1.5	33.5	32.0	64.3	35.1	57.7
153	3	30.1	30.4	64.9	35.9	57.6
155	1	31.2	29.5	56.8	40.6	68.3
155	2	33.6	34.4	65.0	34.6	52.6

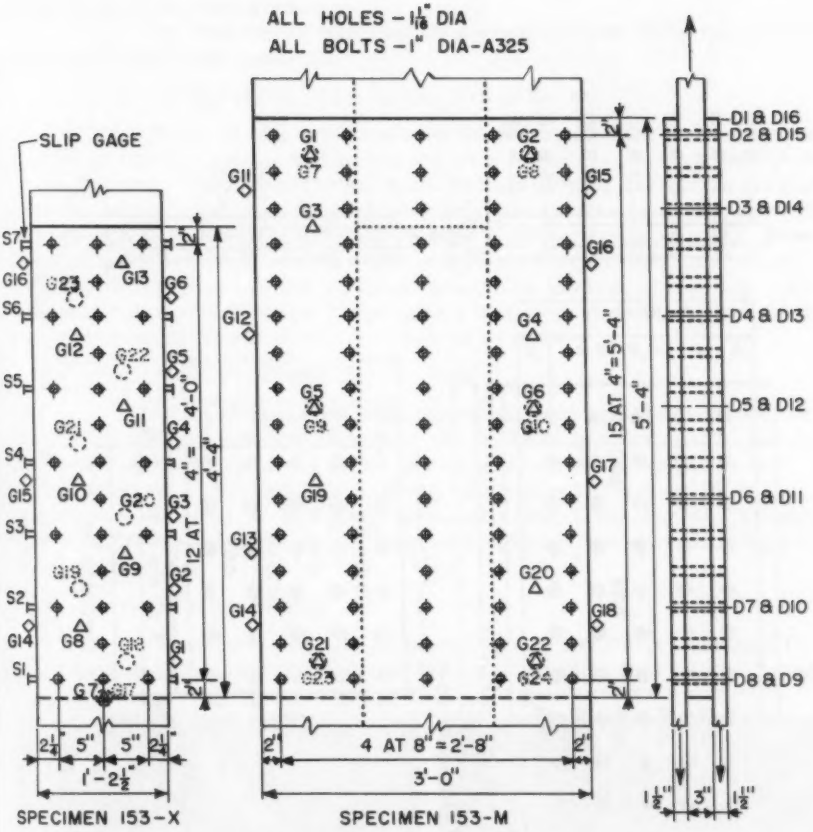

Fig. 6 Specimen 153-M (tension:shear ratio 1:0.73) and 153-X (tension:shear ratio 1:0.8). Faying surface: mill scale.

Fig. 7 Specimen 155-N (tension:shear ratio 1:0.78) and 155-Y (tension: shear ratio 1:0.71). Faying surface mill scale.

The bolts were of 1-in. diameter high tensile type meeting the ASTM specification A-325, and marked P, in 17/16-in. diameter holes.

Before assembling the joints all faying surfaces were properly cleaned from rust and grease.

The bolts were tightened by a complete turn of the nut, starting from the finger-tight condition.

b. Tightening the bolts by one turn of the nut.

Punch marks were made on the nuts and on the ends of the bolt shanks so that the marks for the bolt lay on a radial line through the center of the bolt end. Thus when the mark on the nut traveled 360^0 and came back onto the same radial line, a complete turn of the nut was indicated. An impact wrench driven by compressed air was used which torqued about 90% of the bolts of Spec. 155-N. The rest of the bolts were torqued by a hand wrench with self-locking socket and the hoist mounted on the testing machine. With that particular procedure the torquing of a bolt from finger tight position to a complete turn of the nut took from 12 to 20 seconds.

c. Measurement of slip and strain.

When all bolts had been tightened, dials and strain gages were fitted on the specimen. For specimen 155-N, 14 dials were used to measure slips and 12 SR-4 Type A-11 gages were cemented to the lap plates to measure strains. For specimen 153-M, 16 dials and 24 gages (16 on lap plates, 8 on edges of the butt plate) were used (Fig. 8 and Fig. 9). A Baldwin strain indicator and a Young indicator with switch boxes were used for taking strain readings.

d. Test procedure.

Loads were applied in increments of 200,000 pounds until the full capacity of the machine was reached. Each loading took about 1 minute

Fig. 7 Specimen 155-N (tension:shear ratio 1:0.78) and 155-X (tension: shear ratio 1:0.71). Faying surface mill scale.

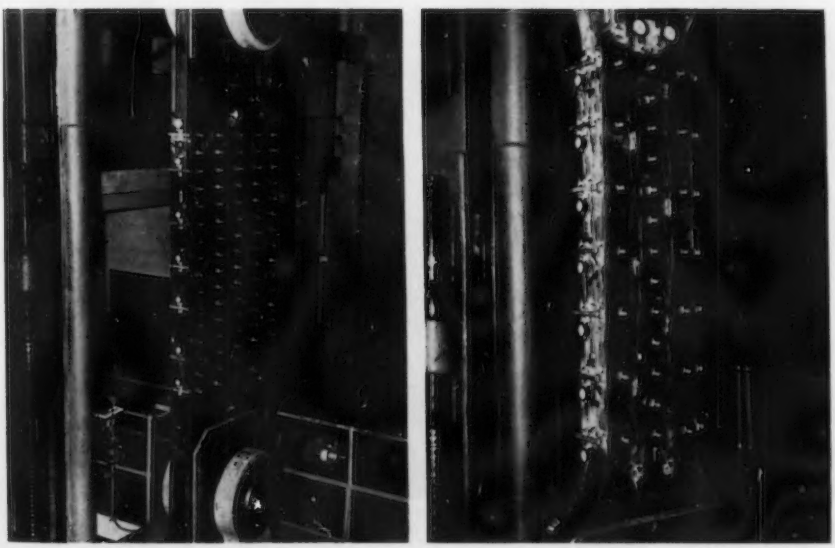

FIG 8-SPECIMENS 153-M (LEFT) AND 153-X READY FOR TEST

and was maintained while readings of dials and gages were taken. This took about 6 minutes in average. The higher loads (over 2,000,000 lbs.) were maintained at each step for an additional 10 minutes so that any significant change in slip or strain which occurred within the short period of time after loading might be discovered. The load was released in steps of 400,000 pounds.

2. Test to Fracture of 27- and 30-bolt Joints 153-X and 155-Y.

a. Preparation of specimens.

The joints tested are specimens 153-X and 155-Y which are shown on Figs. 6, 7, 8 and 9. The properties of the two specimens are listed in Table IIIA.

Specimen 153-X was cut from specimen 153-M, and specimen 155-Y was cut from specimen 155-N. The 1-in. diameter bolts were of ASTM type A-325, marked C for specimen 153-X and marked P for specimen 155-Y. The specimens 153-X and 155-Y were designed so that they could be broken in the 2,400,000-pound testing machine.

The procedure used to prepare, clean and to fit the specimens in the testing machine was identical with the procedure used in the previous slip test.

b. Tightening the bolts.

This procedure was essentially the same as described under paragraph III-A/b.

c. Measurement of strain, slip and elongation.

Dial gages were used for measuring the slip at various rows and the over-all elongation of the specimens. Strains at various points on the

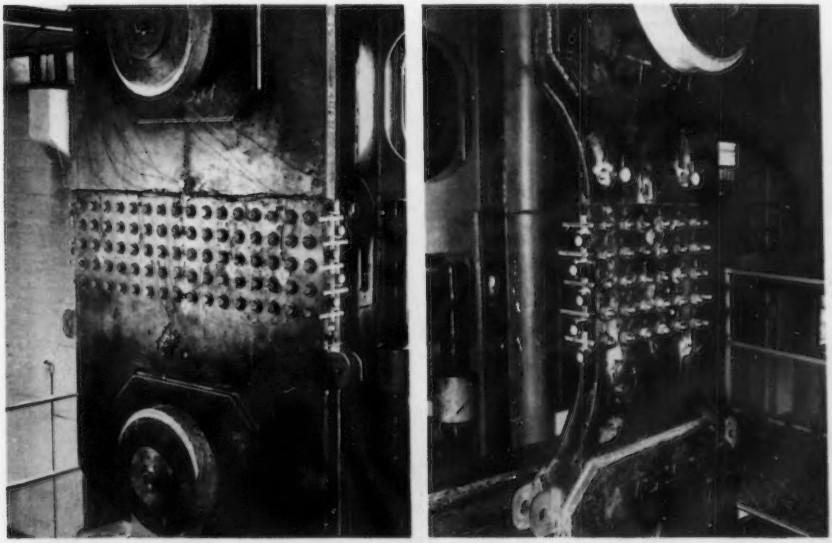

FIG 9-SPECIMENS 155-N (LEFT) AND 155-Y READY FOR TEST

plates were measured by SR-4 Type A-11 gages. A Baldwin strain indicator and Young indicator with switch boxes were used for taking strain readings.

For specimen 155-Y, 12 SR-4 gages were used. In this specimen, which had five rows of bolts, the slip was measured at each row by dial gages fixed at either end. Over-all elongation of the specimen was measured by four dial gages, two on each side. These dial gages were attached to the butt plate at the top end of the joint. They had a wire attachment extending to the other end of the joint and firmly clamped there to the lap plates. The arrangement of all gages can be seen on Fig. 9.

Specimen 153-X had 23 SR-4 gages. The slip was measured for each alternate row with 14 dial gages. For overall elongation a similar arrangement was used as in specimen 155-Y above. The arrangement of these gages can be seen in Fig. 8.

d. Test procedure.

Loads were applied in increments of 100,000 pounds. All dial and strain readings were taken at each increment of load up to the yield point. Each loading took about one minute and the reading of gages took from 6 to 8 minutes approximately. Whenever a major slip occurred the loading was stopped and readings taken. Reading of SR-4 gages were discontinued when the specimen started to yield. Slip readings were taken until all the bolts were in bearing. Dials for slip readings were removed before breaking the joint. Overall elongation was measured for the entire duration of the test.

3. Calibration of bolts.

a. The calibration of bolts.

In order to find the clamping force produced by one turn of the nut, load-elongation curves were drawn for each kind of bolt.

Two bolts of each kind were tested and the load-elongation curves for the bolts used in these specimens are given in Fig. 10.

b. The determination of the clamping force.

To compare slip characteristics for different types of joints tightened by the one-turn-of-the-nut method does not appear to be a scientifically rigorous approach. Different grip lengths give different clamping forces when the nut is given one complete turn. The clamping force also differs with different batches of bolts. We may only expect a certain average clamping force by this method as all the bolts yield with elongation resulting from one full turn of the nut.

One complete turn of the nut does not give the bolts an elongation equal to the pitch of the bolt thread. The actual elongation is less than this and differs considerably for individual bolts. But once the bolt yields, the curve becomes considerably flat. This means that even if the elongation produced by turning the nut varies within 50% or more in this range, the clamping force varies only by about 12.5% of the 0.2% yield value depending on the shape of the curve. In other words, once yielding occurred, variations in the angle of rotation of the nut would not influence significantly the clamping force.

Thus the average clamping forces are 72,500 lbs. for P bolts of 4.38-in. grip which were used for specimens 155-N and 155-Y; 70,000 lbs. for P bolts of 6.38-in. grip which were used for specimen 153-M, and

69,000 lbs. for C bolts of 6.38-in. grip which were used for specimen 153-X. The adoption of these average values seems to be consistent with the precision obtainable in the one-turn-of-the-nut method.

B. Data and Result of the Slip Test of the 80-Bolt Joints.

The two specimens had been previously tested by torquing the bolts to the required minimum bolt tension[3] and the results analyzed.[7] The present second investigation was done in the same general way except the bolts were tightened by the one-turn-of-the-nut method.[6]

1. Slip Characteristics.

a. The general development of slip.

The joints slipped in the same general manner as when the bolts were tightened to the required minimum bolt tension,[7] but apparently the higher clamping force reduced the slippage considerably for a given load. Figs. 11 and 12 show the averages of two slip gages on each gage line for specimen 153-M and specimen 155-N, respectively, at different loads. It was found[7] that the specimens 153 and 155 under required minimum bolt tension had their major slips at about 2000 and 1500 kips, corresponding to nominal bolt shears of 15.91 and 11.92 ksi, and coefficients of friction of 0.30 and 0.22. By the one-turn-of-the-nut method, specimen 153-M had its first major slip at 2406-kip load, corresponding to a nominal bolt shear of 19.15 ksi, and a coefficient of friction of 0.215 while specimen 155-N showed no major slip at all within the capacity of the machine. The coefficient of friction for the first

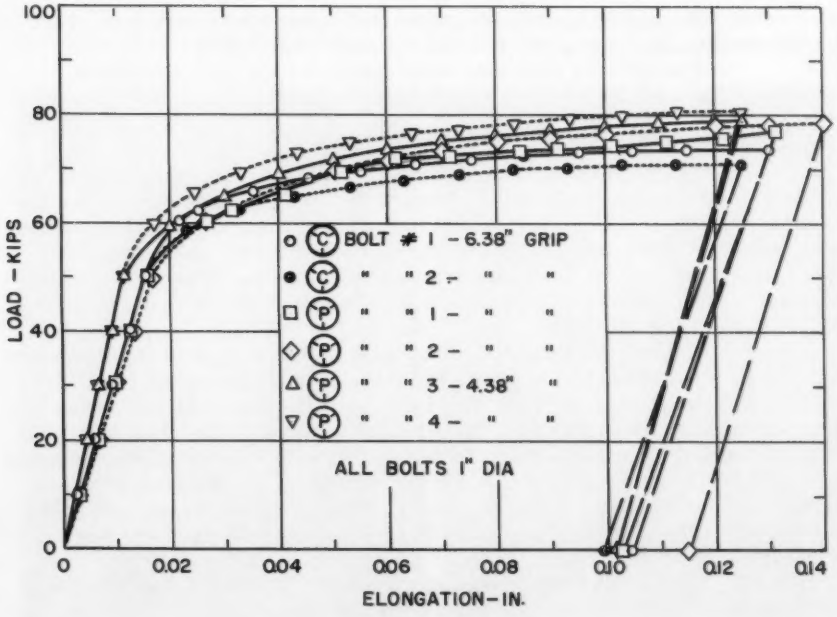

Fig. 10 Calibration curves of 1-inch diameter high tensile bolts used in Specimens 153-M, 155-N, 153X and 155-Y.

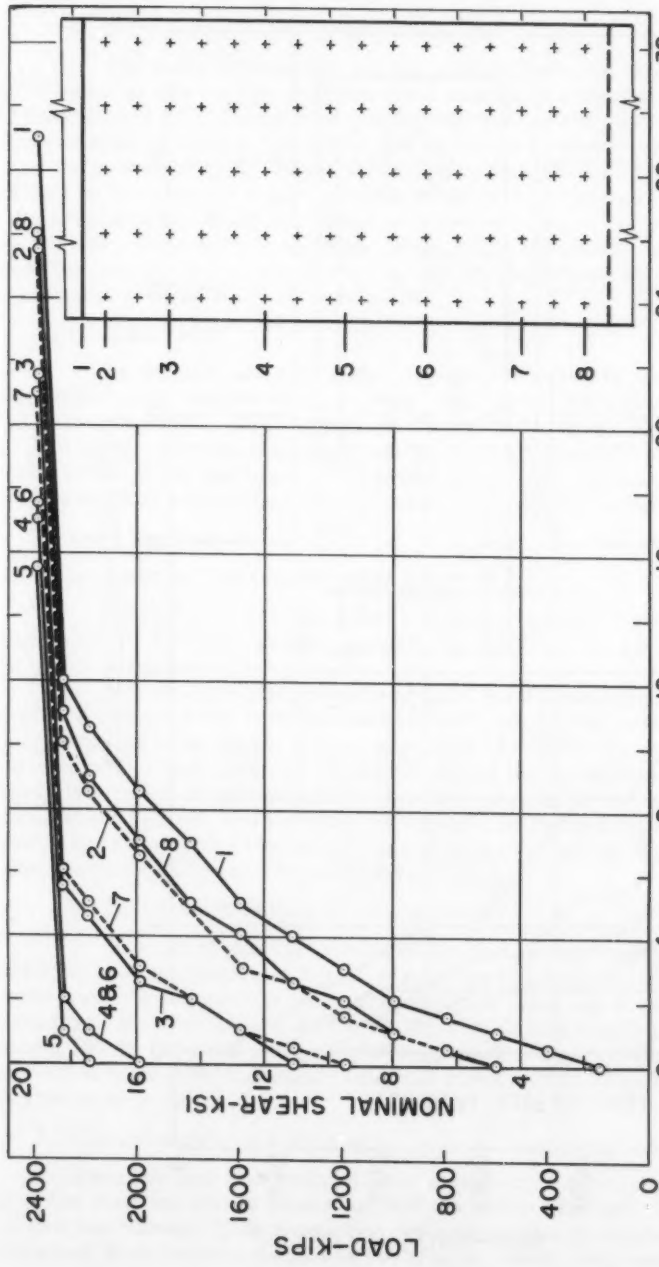

FIG 11. SLIP IN SPECIMEN 153-M

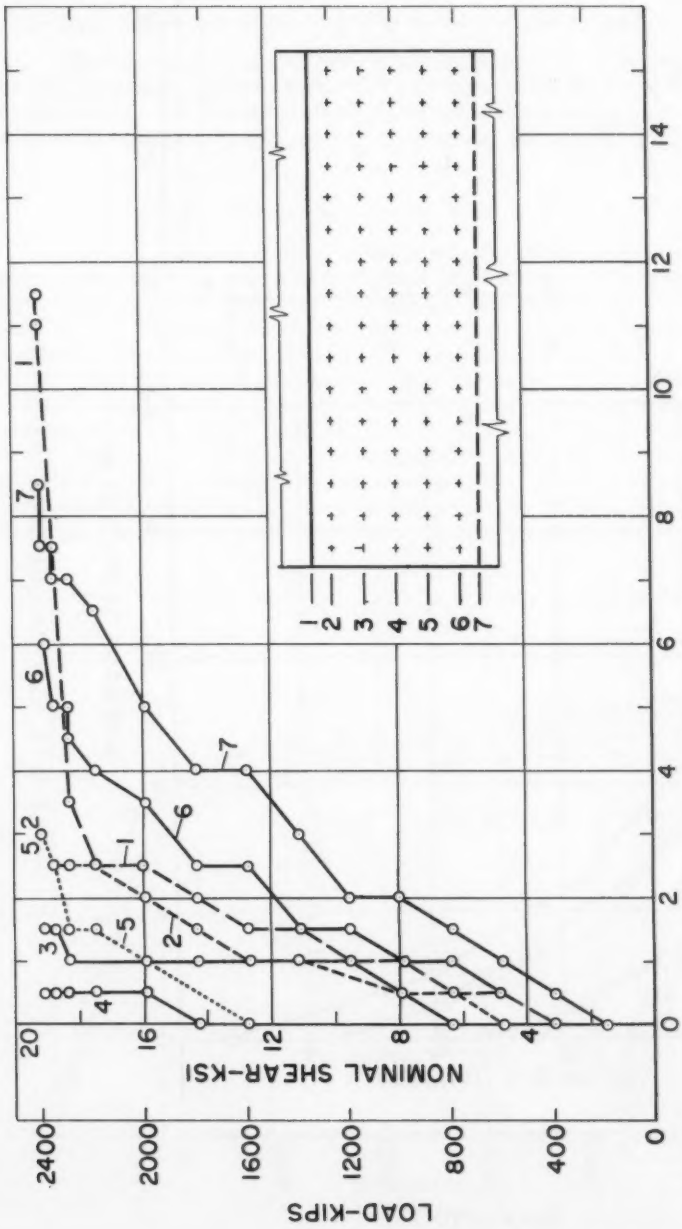

FIG 12. SLIP IN SPECIMEN 155-N

major slip of specimen 155-N is therefore above 0.207, at a nominal shear greater than 19.10.

b. Distribution of slip among rows.

The rows of holes nearest the applied loads, as in the previous tests,[7] began to slip earliest and also have maximum amounts of slip at any load (Figs. 11 and 12). This means that the applied load is picked up only by the two end rows of bolts at low loads, but is shared gradually by interior rows as the load increases. Based on the assumption that the fraction of load taken up by each row is proportional to the slip of that row, it is found that for specimen 153-M the distribution of load in rows is symmetrical with respect to the middle section at higher loads, while for specimen 155-N the distribution has never been symmetrical. No definite trend can be pointed out as the test covered only two specimens.

c. Major slip.

At the first and only major slip that occurred in specimen 153-M, all rows of holes slipped about the same amount—in the order of 1/64-in. or 1/4 of the clearance. Strain readings showed little change at the major slip. It is certain that the bolts were not yet in bearing at this stage. Previous tests of the same specimens[7] showed that the slip might have gone as far as one-tenth of an inch before all the bolts actually came into bearing.

2. Load Partition.

a. Load partition in individual plates.

Comparison with the results of previous tests[7] show that the first row of bolts took up a greater percentage of load in the present test than in the previous ones.

The load partition calculated from strain readings seems to be somewhat erratic and the partition based on uniform friction conforms better to that calculated from slips. But for specimen 153-M the load partition based on strain is very close to that calculated on the assumption of uniform friction. Whether it is true that the longer the joint, the more uniform is the friction between plates, has yet to be ascertained by further tests. However, it should not be concluded that calculation of load partition by slip is more reliable than calculation of load partition by strain.

b. Distribution of load in 2 lap plates.

Slip and strain data show that the distribution of the applied load between the two lap plates may not be exactly equal. For specimen 153-M, the rear plate slipped slightly more than the front plate and consequently took less load than the front plate. Strain readings show this consistently. For specimen 155-N, the front plate slipped far less than the rear plate but jumped up at about 2300-kip load. This was accompanied by a slight reduction of strains in most gages.

3. Slip and Strain on Unloading.

When the load was reduced from maximum to zero, the slips of most of the rows recovered somewhat, and the strain readings did not return to the original values. This means that the measured slip consists of not only a permanent slide between the plates but also an elastic deformation of the

ST 3

plate due to the applied load. Investigation of the slip and strain curves of specimen 153-M discloses a clear relation between the behavior of slip and strain on releasing the load. The apparent slips of the end rows evidently have a greater fraction resulting from elastic deformation than those of the middle rows have. The readings for end sections depart more from the original values on reducing the load than those for the middle sections do, the departure being positive for sections near one end of the plate and negative for sections near the other end.

C. Data and Results of the Fracture Test of the 27- and 30-Bolt Joints 153-X and 155-Y.

1. Slip Characteristics.

a. The general development of slip.

The joint slipped in the same general manner as in the previous test. Figs. 13 and 14 show the averages of two slip gages on each gage line in the joints at different loads. The first major slip was noticed in specimens 153-X and 155-Y at loads of 999 kips and 742 kips, which correspond to a nominal shear of 23.6 and 15.73 ksi and a nominal coefficient of friction of 0.268 and 0.170. Before the first major slip was obtained specimen 155-Y slipped a very small amount at a load of 640 kips corresponding to a nominal bolt shear of 13.6 ksi and a nominal coefficient of friction of 0.146.

b. Distribution of slip among rows.

In accordance with our expectations the rows at the ends, that is, rows nearest to the applied load, start slipping first and show the maximum slip at any stage. This can be seen in Figs. 13 and 14 for specimens 153-X and 155-Y. In specimen 155-Y, row 2 shows less slip than the middle row for a great length of time, and seems to be taking minimum share of the load. Whereas in specimen 153-X the middle row shows no slip until there is a major slip in the whole specimen; it also shows minimum slip all the time. Specimen 153-X shows some symmetry in the distribution of load with respect to the middle row.

2. Strain Variations.

The strain under different loadings in the lap plates and in the center plate of specimen 153-X as measured by SR-4 gages can be seen in Fig. 15. Similarly, Fig. 16 gives strains in the lap plates and in the center plate of specimen 155-Y. These strain values can be interpreted as the average strains in the particular sections of the plates at the different loads. All strain gages that lie within the joint show consistently greater values of strains in sections that lie nearer to the applied loads. The gages which lie outside the first row of bolts however show a much smaller value of strain.

3. Load Partition.

a. Load partition in individual plates.

Percentage of applied load present at different sections of the plates and the load transferred at each row of bolts have been calculated in two different ways. From slip data the load partition has been calculated on the assumption that the transfer of load in each row is proportional to the average slip in the row. Load partition has also been calculated from strain

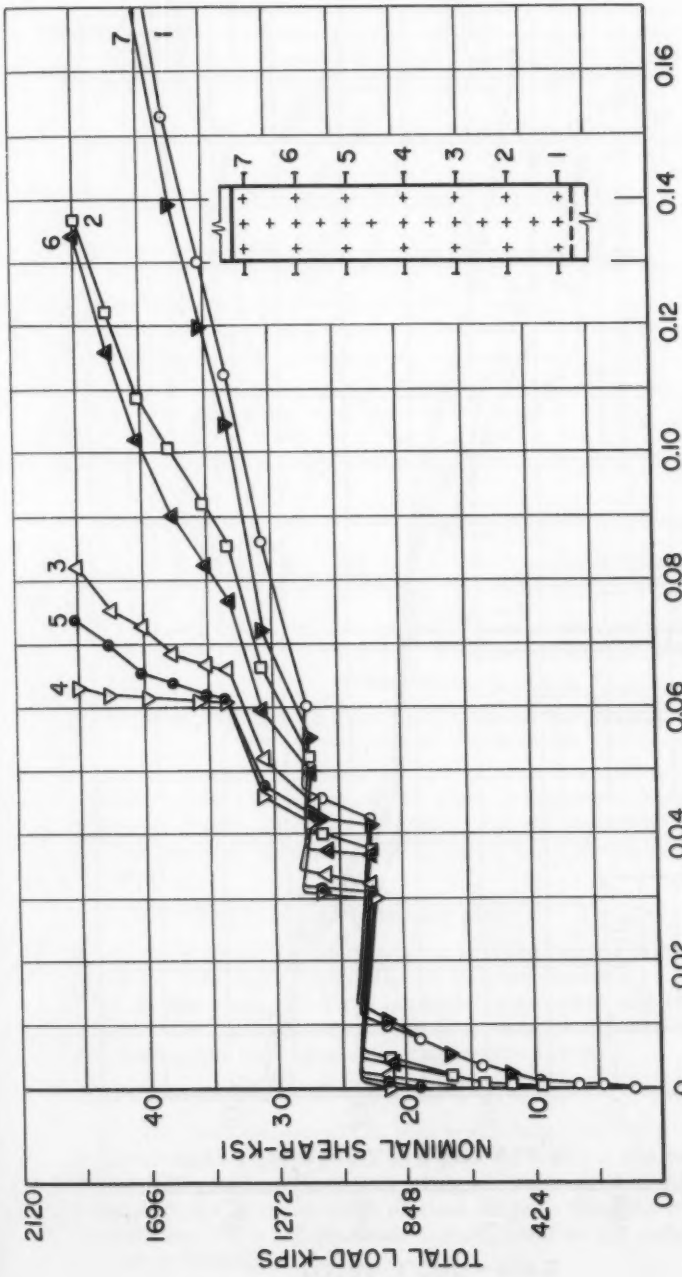

FIG 13. SLIP IN SPECIMEN 153-X

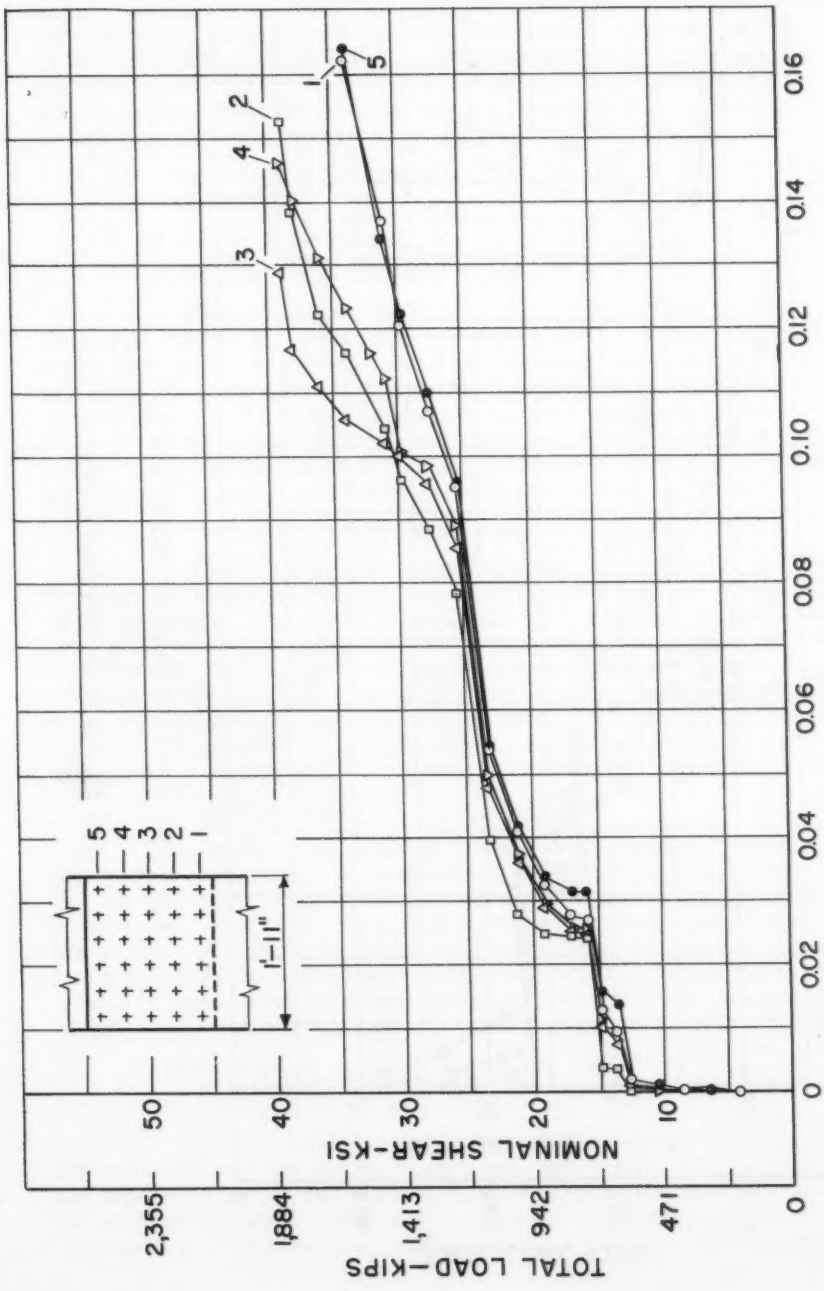

FIG. 14. SLIP IN SPECIMEN 155-Y

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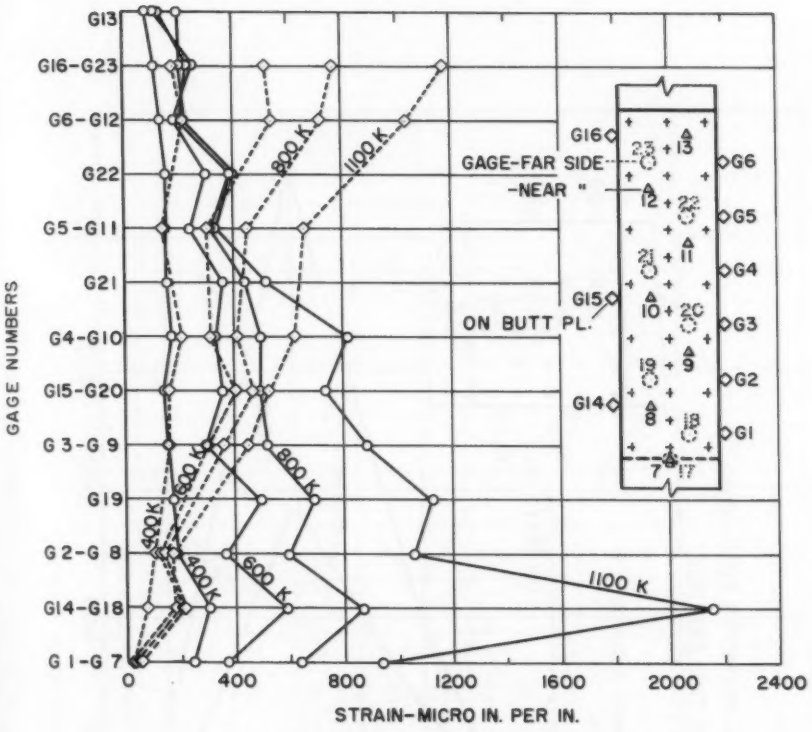

Fig. 15 Specimen 153-X. Strains in the lap plates and in the butt plate at various loads.

readings, assuming that the stress in the plate is uniformly distributed over the whole area of cross section of the plate. Fig. 17 shows the typical load partition in the plates of specimen 153-X, and Fig. 18 shows the same for specimen 155-Y.

b. Distribution of load in two lap plates.

The slip and strain data show that the two lap plates do not take exactly the same load, and slip by slightly varying amounts.

In specimen 155-Y the lap plate on the west side slipped more than the lap plate on the east side. This difference of slip was very much less pronounced in the two lap plates of specimen 153-X.

c. Load partition and slip in the long joint.

The development of the slip in the long joint is in a way different from that observed in a short joint. In a joint with only a few rows of bolts slip develops faster and one jerk may bring the bolts close to bearing. In a long joint slip occurs in much more distinct stages. The slip in both cases starts from the end rows and proceeds inward towards the center of the joint as the load is increased.

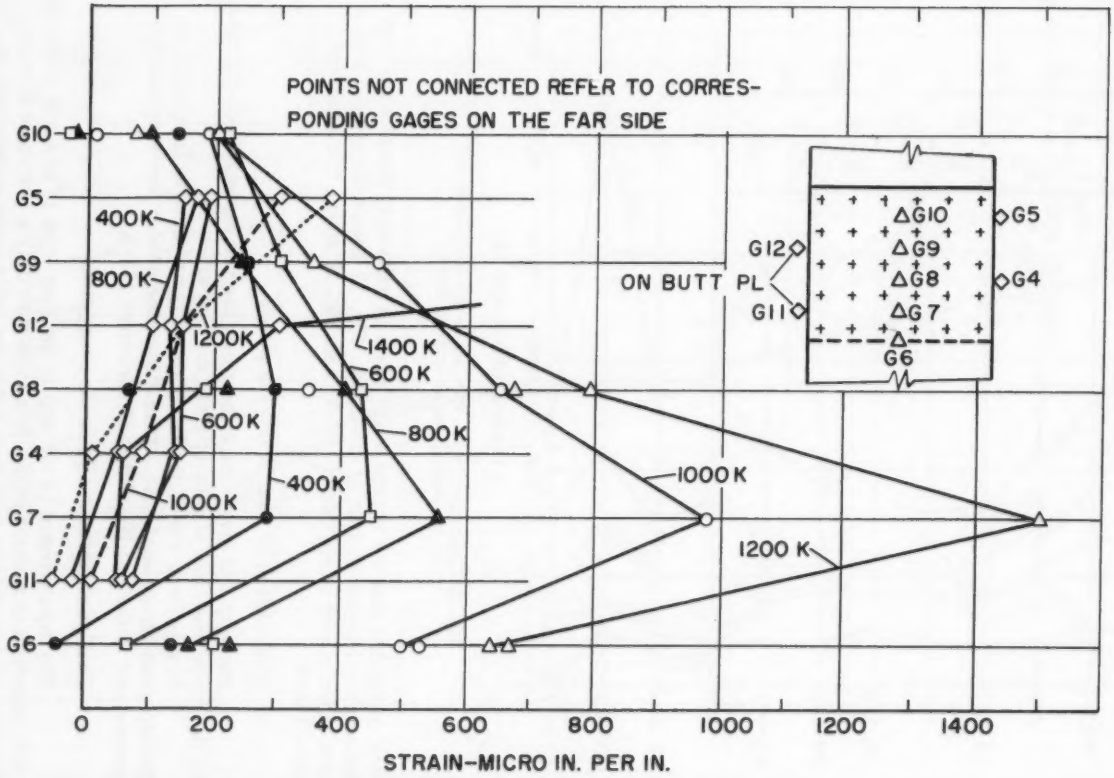

Fig. 16 Specimen 155-Y. Strains in the lap plates and in the butt plate at various loads.

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Fig. 16 Specimen 155-Y. Strains in the lap plates and in the butt plate at various loads.

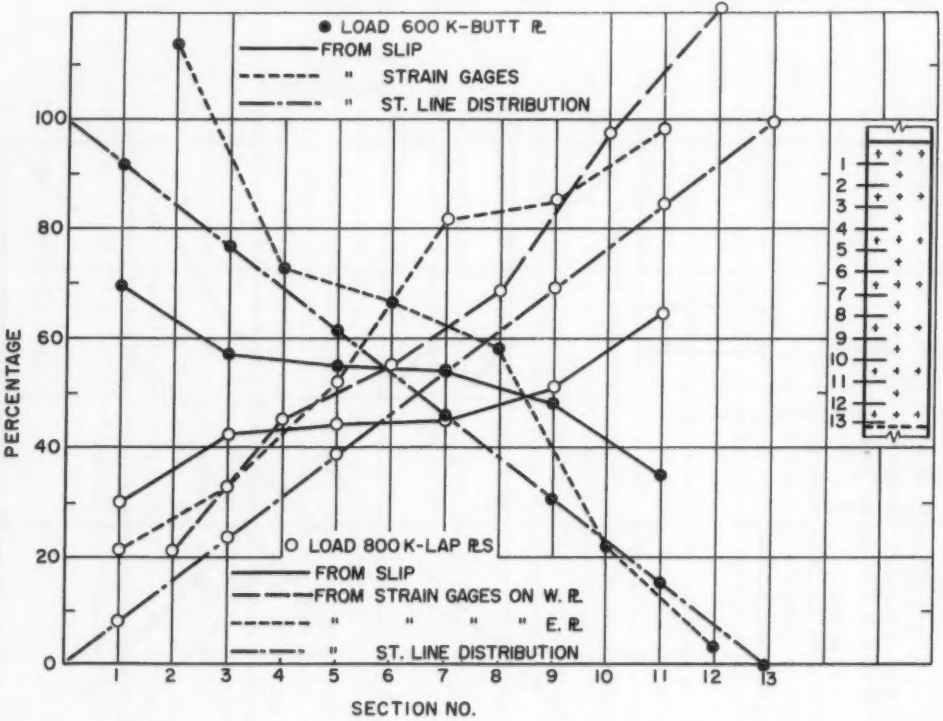

Fig. 17 Specimen 153-X. Typical load partition computed on various assumptions.

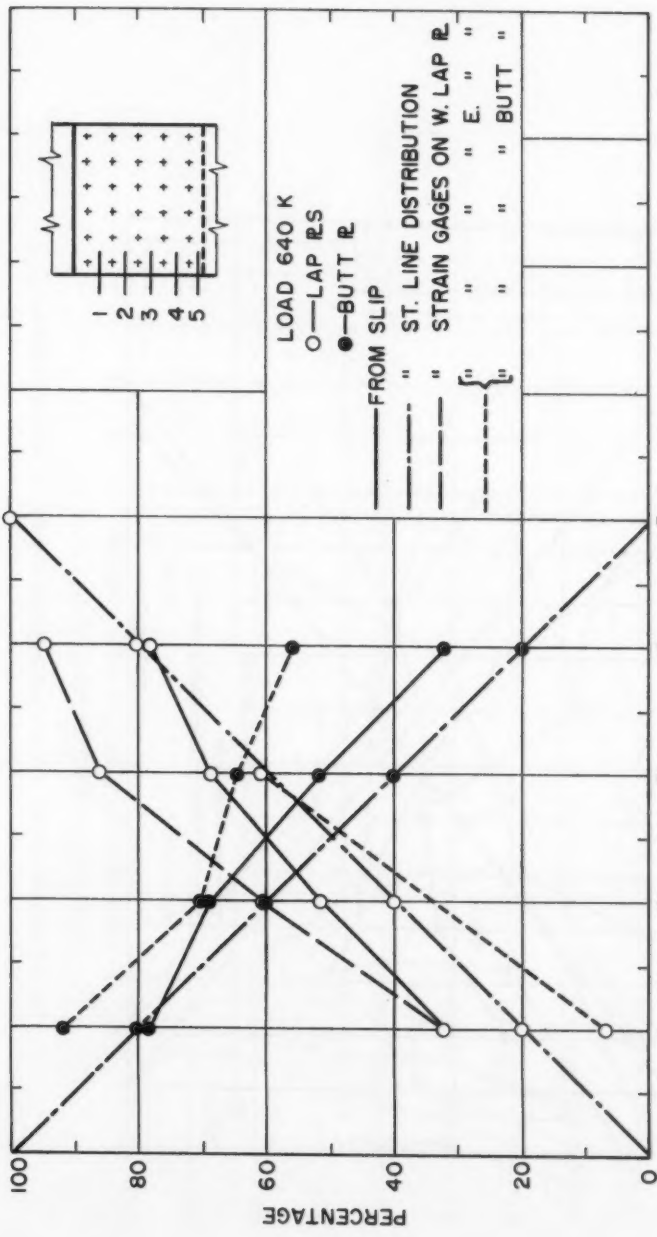

Fig. 18 Specimen 155-Y. Typical load partition computed on various assumptions.

Fig. 18 Specimen 155-Y. Typical load partition computed on various assumptions.

The load applied to the joint is carried by the center plate up to the edge of the lap plates. Past this line, assuming uniform friction there should begin a distribution of the load to the two lap plates. In order to accomplish this the three plates should elongate the same amount. This condition obviously can not prevail near the end of the lap places since there the center plate carries full load while the lap plates carry only very small load. Consequently slip between the plates is forcibly started as a result of the difference of strain between the center and the lap plates. This creates an area over which the coefficient of friction suddenly drops to its dynamic value. In this instant the load carried by the area which forcibly slips, will be partially thrown into the parts which did not slip yet. This process closely resembles the phenomenon called slipstock in the study of frictional phenomena.(8)

Slip will continue by this mechanism from the area of one row to another until it comes near the center of the joint. A stage is reached when the load released by the slip of the last part area can not be transferred by those parts of the joint that have not slipped yet. At this instant the whole joint slips.

On completion of the slip the bolts in the end rows go into bearing first. After these bolts have started deforming the bolts in the middle rows go into bearing.

From the above analysis it appears that the load being transferred through a row is not proportional to the amount of slip at that row. An analysis has, however, been carried out on the basis of slip data and given here for comparison with similar values in previous reports.

4. Elongation and Yielding of the Specimens.

The average over-all elongation of the joints has been plotted against each applied load and is shown on Fig. 19.

5. Failure of Specimens.

Contrary to our expectations the specimen 155-Y did not fail in the thicker plate but in the two lap plates (Fig. 20). This was in accordance with the fact that the thinner plates had a lower ultimate strength in this specimen.

Specimen 153-X failed by shearing of the bolts. At a load of 2,260 kips one end bolt in the bottom row sheared off. The load was dropped to observe what had happened. On increasing the load to 2,259 the two other bolts in the bottom row sheared off. The load was again dropped and on reloading after the observations two bolts in the top row and the one bolt in the second row from the bottom dropped off at a load of 2,248 kips. On reloading again the last bolt in the top row and the one bolt in the second row from the top sheared off at a load of 2,276 kips. After this all the remaining bolts sheared off at once.

6. The Efficiency of the Joint.

The efficiency of the joints 153-X and 155-Y has been computed in this report by the Relative Gage Method developed by F. W. Schutz, Jr., and N. M. Newmark.(4) This method gives the efficiency of the joint when failure occurs in the plate. Specimen 153-X failed by shearing of bolts. So the efficiency computed here does not check closely with its observed value. In specimen 155-Y failure occurred in the plates and the observed efficiency

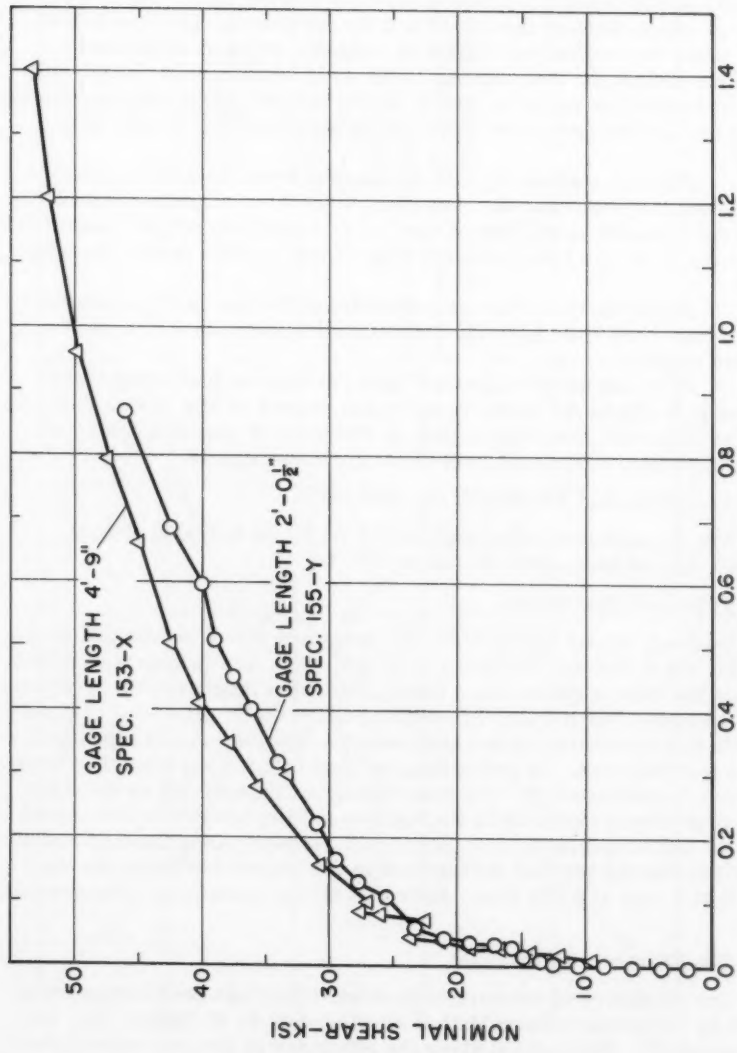

Fig. 19 Average overall elongation of Specimens 153-X and 155-Y.

checks closely with the computed efficiency. The results of this analysis are the following:

> Specimen 153-X had 2,276 kips observed and 2,401 computed ultimate load, 81 per cent observed and 87 per cent computed efficiency. Specimen 155-Y had 2,164 kips observed and 2,080 computed ultimate load, 83 per cent observed and 79.6 per cent computed efficiency.

D. Probability Test of C_6 Type 4-Bolt Joints.

In order to discover any effects of the one-turn-of-the-nut method of tightening the bolts upon the nominal coefficient of friction, a special set of tests were run on 4-bolt joints.

1. Preparation of Specimens.

The C_6 type specimens were made of ASTM A 7-51T steel plates, 1/2-in. and 1-in. thick. Thirteen double assemblies were made which provided specimens for 26 tests.

a. Tightening the bolts.

The bolts used for these small joints were 1-in. in diameter and 4-in. long, and were marked BIS on the head. The holes were of 17/16-in. diameter. One-half turn has been used since previous tests showed that the nuts will strip at about 3/4 turn, the grip being only 2-in. Calibration test of the bolts showed that the full clamping capacity of the bolt is developed by one-half turn.

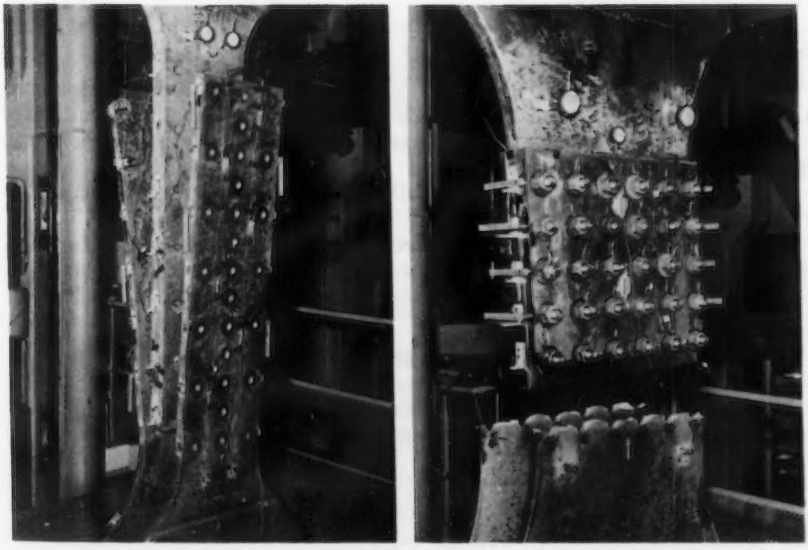

FIG 20-SPECIMENS 153-X (LEFT) AND 155-Y AFTER TEST

Fig. 19 Average overall elongation of Specimens 153-X and 155-Y.

b. Test procedure.

The ends of the assembled specimen were bolted to two prefabricated connecting pieces. The whole assembly was lifted to and fixed in the 300,000-lb. testing machine. Two dials were used, one on each edge of the specimen to measure slip. Load was applied in 10-kip increments and the corresponding slip values taken until the evidence of bearing was noticed. The sudden drop of load indicated the occurrence of a major slip.

2. Test Data and Results.

a. The general development of slip.

The joints slipped in the same general manner as when the bolts were tightened to the required minimum bolt tension,[3,9] but apparently the higher clamping force reduced the slippage considerably for a given load.

Fig. 21 shows the load-slip curves of all the 26 C_6 type specimens. The average value of nominal bolt shear at which major slip occurred is 21.67 ksi, corresponding to an applied load of 136.2 kips and coefficient of friction of 0.284 when the bolt tension is 60 kips. Previous test results[3,9] showed that under the minimum required bolt tension of 42.5 kips, major slip occurred in similar C_6 type joints at average nominal bolt shears of 13.8 and 15.3 ksi corresponding to coefficients of friction 0.25 and 0.29; and under 52.5 kips bolt tension, major slip occurred at an average nominal bolt shear of 21.8 ksi corresponding to coefficient of friction 0.33. The value 21.8 ksi for 52.5-kip bolt tension seems to upset the general trend that major slip occurs at higher nominal shear value when bolts have higher tension. But this should certainly be attributed to the fact that only 3 specimens were tested under 52.5-kip bolt tension.[9]

In the test of the 4-bolt specimens, four different ways of slipping were noticed.

(1) Slip started gradually, jumped suddenly at a certain load value forming a so-called major slip which was indicated by a sudden drop of load with or without an accompanying loud report. Then slip increased gradually again as the load increased.

(2) Slip progressed gradually all the way, no drop of load occurred within the load range.

(3) Slip appeared in the same general way as described in (1). Load dropped twice indicating two major slips.

(4) Same as (1) and (3), except load dropped three times indicating three major slips.

Three C_6 type specimens (Nos. 2, 9, 10) were assembled with the center lines of the lap and butt plates inclined at a small angle of about $1\text{-}1/2^{\circ}$. No particular behavior in slipping can be attributed to the misalignment of the specimen so far as these few tests are concerned.

b. Amount of slip.

The amount of major slips in the 4-bolt joints varied from 1/10 to 1/2 the clearance, but the majority was around 1/64-in., or 1/4 the clearance. The maximum amount of slip (average the readings of 2 dials) at 170-kip load was 0.069-in. (Fig. 21), greater than the nominal clearance of the hole. Actually many joints had been in at least partial bearing after major

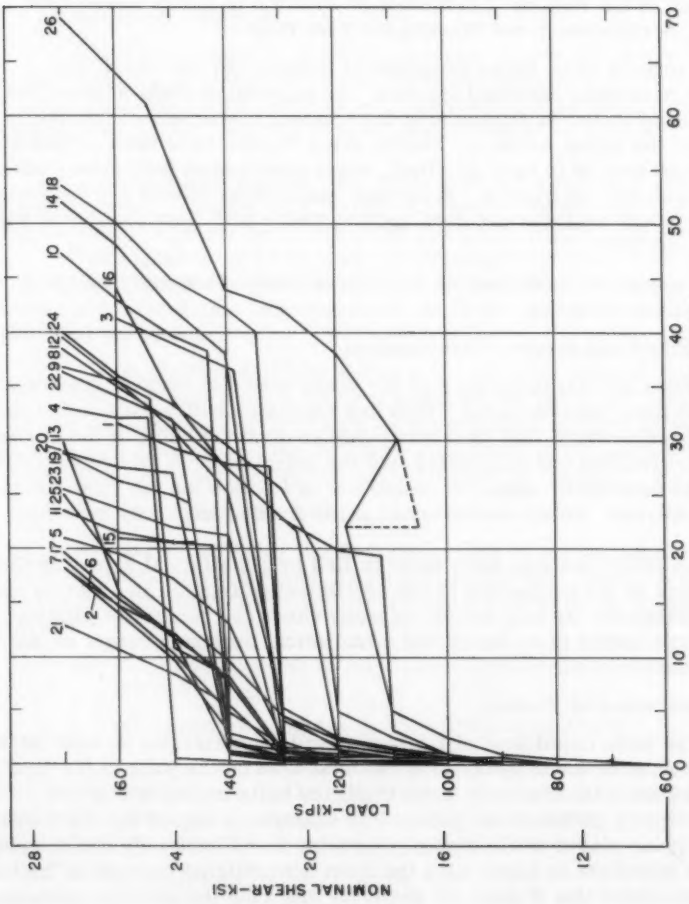

Fig. 21 Slip in C_6 Type Specimens.

slips occurred, but the reading on the slip dials continued to increase probably because the contacting parts of the metal were deformed.

c. Probability of slip.

In all types of similar joints, there is always a certain range of load value within which slip will most probably occur. The probability analysis of slip for the C_6 type joints indicates that major slip is most likely to occur when the nominal bolt shear is between 22 and 23 ksi.

III. The Study of the Nominal Coefficient of Friction in Joints Based upon Illinois, Northwestern and Washington Test Data.

Sixteen reports from the universities of Illinois, Northwestern and Washington have been searched for data. An exhaustive study showed that the most important factor in determining the nominal coefficient of friction is the treatment of the faying surface. Twelve other factors have been considered, some of which proved to have no effect, some others have only minor effects upon the coefficient of friction. In certain cases these effects are well definable. A statistical analysis has been applied where sufficient number of data could be found.(10)

A. The Apparent Coefficient of Friction in Joints Fastened by High Tensile Steel Bolts.

1. Object and Scope of Investigation.

The slip characteristics of the joints tested in various structural laboratories have been analyzed. This has been done with a view to investigating the factors which may have some influence on the value of the nominal coefficient of friction and if possible find the extent of such influence. Only the values of nominal or apparent coefficient of friction for the first major slip were analyzed. Joints tested under static loading have only been considered.

An effort has also been made to find by a statistical approach the lack of control in the estimation of the coefficient of friction in order to enable us to determine its safe value. In many cases the number of joints of a particular type tested is so small that a statistical approach cannot be successfully used.

2. Definition of Terms.

The term coefficient of friction of spliced materials as used here is the nominal value found by dividing the total load on the joint at the first major slip by the total clamping force of all the bolts multiplied by the number of slipping planes in the joint. For example, a double lap joint would have two slipping planes while a single lap joint would have only one slipping plane. This definition is based upon the most conventional concept of friction and it is understood that it does not perfectly describe the complex phenomena resulting in measurable slip in a joint. This measured slip results in part from deformations of the joined plates and in part from their actual relative motion. The two types of slip are interrelated and their selection would neither be practicable nor practical. Observations made on large joints(6,7) indicate that the first slip measured during the test of a joint is mainly the result of combined causes and only the so-called first major slip is unequivocally a slip in which the whole of the joined plates take part in the motion.

The term first major slip refers to the first such relative motion of the joined plates, in which one whole plate unmistakably takes part. This is represented on graphical presentations of slip-force curves as a definite horizontal portion. On larger joints gages in all positions simultaneously indicate such slips. This first major slip might be the one which brings all or part of the connectors into bearing. Defining our nominal coefficient of friction in connection with this first major slip seems to provide it with more definite technical meaning.

3. Method of Analysis.

A list was prepared of all the specimen joints that were relevant to this report and the properties of these joints and their slip characteristics were recorded. This led to such an accumulation of data which cannot be presented here. It has been made available in reference 10 in a condensed form.

The conclusions of each progress report were also studied. This was done in order to enable us to select the factors influencing the coefficient of friction of a joint from the factors that did not definitely influence it.

The significance of each factor affecting the value of the nominal coefficient of friction was then studied. These factors were then chosen as variables and a tabulation has been made of all the joints similar in respect to all variables but one.

The statistical analysis followed the practice of using frequency charts and control diagrams to study the lack of control in the values of the coefficient of friction. The available tests were used in three different groups for this approach. This was done in order to arrive at a probable value that can be recommended for the coefficient of friction.

4. Factors Affecting Friction.

Any factor affecting the first major slip in a joint necessarily reflects in the value of the nominal coefficient of friction. Some factors seem to have a more definite effect than others, thus it is in order to group them as follows.

a. Factors that seem to affect significantly the coefficient of friction.

(1) Condition of faying surface[9,11,12,13,14,15]

From the laws of friction it directly follows that the coefficient of friction depends on the roughness of the surface. Consequently the condition of faying surface makes all the difference in the value of the nominal coefficient of friction. It is the most important factor.

(2) Bolt tension[11,12,13,16]

It has been stated[11] that the nominal coefficient of friction is independent of the clamping force of the bolts. This seems in line with the generally known laws of friction. In (16) it is stated, "The coefficient of friction was independent of the clamping force of the fasteners, provided the tensile stress on the net cross-section area did not exceed approximately one-half of the upper yield point stress of the plates in the joint". Which means that beyond a particular value of bolt tension the design is governed by the net area in tension. Due to stress concentration the plate near the bolt holes

yields at a stress in the net section corresponding to half the value of the upper yield point.

In view of this consideration the nominal coefficient of friction in a joint made by high tensile bolts can be affected by bolt tension.

(3) Tension:shear ratio (12,13)

The tension:shear ratio seems to have an effect on the nominal coefficient of friction. This is probably due to the fact that a greater stress in the net section reduces the clamping force in the bolts due to Poisson's ratio effect and also causes the plate to yield earlier near the bolt holes.

(4) Type and size of joint (9,7,6,13)

The results of the tests described in the above references seem to show that the type and size of joint have some influence on the nominal coefficient of friction. In the lap plate joint probably slight bending occurs and friction is called upon only to take a component of the total load. (13) In the case of joints with several rows, the end rows slip first.

(5) Duration of load (12)

A tentative conclusion mentioned in (13) shows that duration of load has an effect on the value of the nominal coefficient of friction. This can also be seen from results of tests. Sustained load probably creates some creep along the surfaces in contact making them smoother.

(6) Misalignment of holes.

Misalignment of holes can affect the nominal value of coefficient of friction because some bolts may be in bearing (fully or partly) to start with. Unless the plate or the bolt deforms at that place slip will not occur.

(7) Temperature in the case of painted surfaces (2)

The paints harden at very low temperatures and show greater resistance to slip. But this seems to be of no practical importance as the strength is again lost as temperatures rise.

b. Factors that seem to have no significant effect on the coefficient of friction.

(1) Size of faying area (11,12)

The area of surfaces in contact has no effect on the value of the nominal coefficient of friction. This result also seems to be in line with the laws of friction.

(2) Thickness of lap plates (12)

The thickness of lap plates seems to have no effect on the value of the nominal coefficient of friction as long as the average stress in the lap plate does not exceed 60% of the tensile yield point of steel. The effect of this variable is probably negligible because the reduction of bolt tension due to diminishing of the thickness of the plate on account of Poisson's ratio effect is negligible. But it possibly becomes more effective when the portion of the plate near the bolt hole yields.

(3) Clearance in bolt holes[12]

There appears to be no effect due to the clearance in the bolt holes on the nominal coefficient of friction as long as the bolts are not in bearing to start with.

(4) Type of loading (tension or compression)[16,12]

It is mentioned in (16) that there seems to be some effect of the type of loading on the value of the nominal coefficient of friction. But this has been contradicted in (12). Probably the effect, if any, is due to the Poisson's ratio effect in the plate.

(5) Bolt grip[9,17]

It is stated in (9) that the length of the grip does not seem to have any influence on the coefficient of friction. In the conclusions of (17) no mention has been made of any influence of the bolt grip; nor do the results show any great effect of this factor.

5. The Effect of Various Factors Upon the Nominal Coefficient of Friction.

a. Condition of faying surface.

Many groups of joints have been tested which were similar in all respects except in the condition of their faying surface. A list of all the specimens which fall in this group can be found in reference 10.

In general it is evident that any kind of paint tends to act as a lubricant and reduces the coefficient of friction. The value of the nominal coefficient of friction seems to be rather small in the case of surfaces painted with red lead and varnish. The values of the nominal coefficient of friction in the case of lacquered surfaces show a particular behavior. In one of the series of tests a specimen with lacquered surface showed greater coefficient of friction than similar specimens with mill scale surfaces. Sand blasting seems to make the surfaces rough and increases the coefficient of friction.

In general, mill scale surfaces from which loose scales have been removed seem to show a reasonable roughness of the surface and a greater nominal coefficient of friction. This means that if the surfaces can not be made rough then at least no kind of paint should be used on their faying surfaces.

The faying surface treatments are listed as follows in the order of decreasing coefficient of friction using average values:

(1) Rusted and wire brushed (0.51)
(2) Rusted (0.48)
(3) Sand blasted (0.47)
(4) Mill scale (0.30)
(5) Lacquered (0.28)
(6) Galvanized (0.26)
(7) Aluminum suspension paint (0.17)
(8) Varnished (0.14)
(9) Rust prevention paint (0.11)
(10) Black graphite paint (0.09)
(11) Red lead (0.065)

To these the following can be added from more recent tests:

(1) Mill scale, flame cleaned and wire brushed (0.43)
(2) Red lead, sand blasted (0.42)
(3) Red lead, plates assembled wet (0.31)
(4) Red lead, flame cleaned (0.22)
(5) Second coat of red lead on a dry one, assembled wet (0.10)

Since only two specimens of each kind have been tested, the results should not be attributed the same weight as was attributed to the first enumerated ones.

b. Bolt tension.

The data which have been used to study the effect of bolt tension on the coefficient of friction fall into two groups, one with tension:shear ratio of 1:0.94 and the other with tension:shear ratio of 1:0.75. They indicate that the value of nominal coefficient of friction is fairly constant up to a certain range of bolt tension. When the bolt tension is increased beyond a particular value (see paragraph 3Aa) the value of nominal coefficient of friction decreases as indicated.

It may be reasonable to infer from this that the value of nominal coefficient of friction decreases at higher values of bolt tension. This decrease is of the order of about 0.07 which is not of such magnitude as to call for our limiting the value of bolt tension.

c. Tension:shear ratio.

Five groups of tests have been used to find the effect of tension:shear ratio.

Only test groups which consist of several tests were included in the plots. It is noticeable that as the tension:shear ratio increases the increase in the nominal coefficient of friction is somewhat steeper at first and tends to level off. The increase is of the order of about 0.06.

d. Type and size of joints.

(1) Joints with 2 or 3 bolts.

A survey of the slip characteristics of lap and butt joints fastened by two or three bolts having an average bolt tension of 35 kips showed that the values of nominal coefficient of friction vary from 0.18 to 0.60 which seems to be rather a large variation. These variations may be due to the fact that the number of bolts and size of joints is too small to average out any variations in the condition of surface, the actual tension achieved in the bolt or any other possible reason. This also seems to be a reason to be more conservative in estimating the coefficient of friction in the smaller joints.

(2) Joints with 4 or 6 bolts.

The 92 joints tested with 4 or 6 bolts show an average coefficient of friction of about 0.31 ranging from a minimum of 0.18 to a maximum of 0.45. In this case, though the average value is lower than in the smaller joints, we do not have such a great variation.

If we consider 51 joints with 4 to 8 bolts having an average bolt tension of 42.5 kips we have the average value of nominal coefficient of friction as 0.337 with a minimum of 0.22 and a maximum of 0.45 which is an even lesser variation.

Twenty-six joints tested with 4 bolts each and having an average bolt tension of 60 kips show an average value of nominal coefficient of friction of 0.284, a minimum of 0.233 and a maximum of 0.322. This is a still smaller variation.

(3) Joints with 20 bolts or more.

Six joints assembled with 20 to 30 bolts show an average value of nominal coefficient of friction of 0.22, a minimum of 0.17 and a maximum of 0.30.

Five joints assembled with 80 bolts show an average value of nominal coefficient of friction of 0.23, a minimum of 0.21 and a maximum of 0.30.

Summing the approximate values of average coefficient of friction in connection with the number of bolts in the joint:

2 or 3 bolts, ave. 0.45 (0.18 - 0.60)
4 to 6 bolts, ave. 0.31 (0.18 - 0.45)
20 and more bolts, ave. 0.22 (0.17 - 0.30)
80 bolts, ave. 0.23 (0.21 - 0.30)

(4) Effect of size.

The number of joints tested having less than 4 bolts or more than 4 bolts is very small to draw any definite conclusions. The average value of the nominal coefficient of friction seems to decrease with the size of joints and so does also the range between the minimum and maximum values of the nominal coefficient of friction. If we regard carefully the study of joints with 4 to 6 bolts it suggests that there is less lack of control of the value of nominal coefficient of friction in joints having higher bolt tension. All the large joints tested have an average bolt tension of 42.5 kips or greater. All the joints tested with 2 or 3 bolts have a bolt tension of 35 kips.

(5) Effect of shape of the joint.

Joints in which the number of rows is more than 4 times the number of bolts in each row seem to have a very small value of nominal coefficient of friction. Three joints[9] having 10 rows of bolts with 2 bolts in each row show an average value of nominal coefficient of friction of 0.22, a minimum of 0.21 and a maximum of 0.23. One joint with 13 rows of bolts having alternate rows of 1 and 3 bolts showed a coefficient of friction of 0.17.

e. Duration of load.

The data on the effect of sustained loading indicate that there is a 20% decrease in the value of the nominal coefficient of friction when tested under such conditions. The observation is based only on a single specimen. However, this would mean that the value of the coefficient of friction should be estimated with care when used in connection with sustained loading.

6. Statistical Analysis.

a. Selection of joints.

We find that the most important factor affecting friction is the type of faying surface. The variation in the value of coefficient of friction becomes so great with the change of faying surface that it does not seem reasonable to group joints with different types of faying surfaces for statistical treatment. The generally accepted practice is to make joints with members

having mill scale surfaces. Joints with mill scale faying surfaces give us the most reasonable average values of coefficient of friction. The available data seem to justify a statistical treatment of these joints only.

b. Analysis of all joints having mill scale surfaces.

An analysis was made of all the 127 joints tested under short time static loading in tension having mill scale surfaces. The cumulative frequency distribution curve for the coefficient of friction as observed in these joints show a considerable variation in the value of coefficient of friction. The value that occurs most frequently, or the mode, is 0.31 as can be found on a frequency hystogram. The central value (or arithmetic mean) of the coefficient of friction is 0.3128.

The control chart for the values of nominal coefficient of friction is shown in Fig. 22. A value of standard deviation from the central value has been computed as suggested in (17).

$$\sigma = \sqrt{\frac{\sum_{i=1}^{n} X_i^2}{n}}$$

where σ = standard deviation

$\bar{X}$ = central value

X_i = value for joint i

n = number of joints

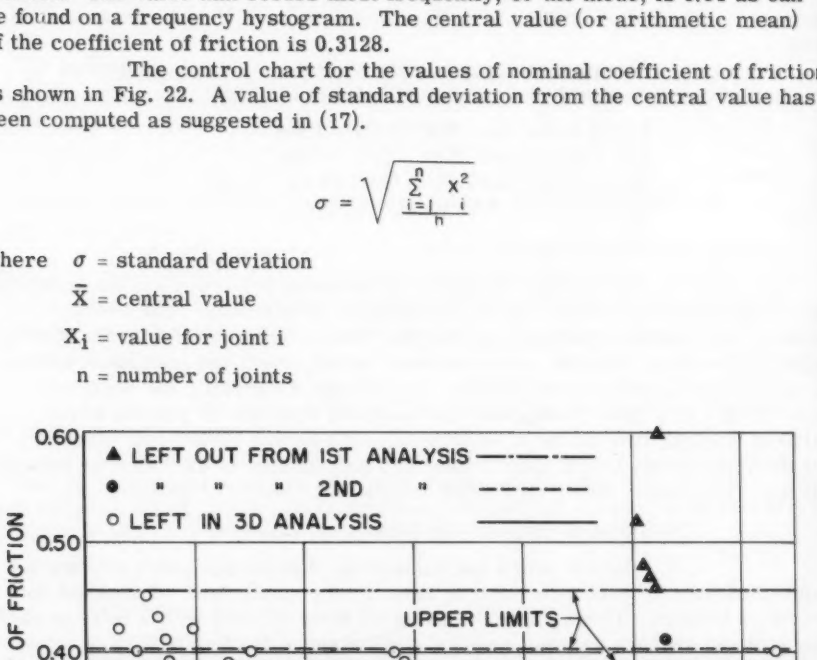

Fig. 22 Control Chart for the Values of Nominal Coefficient of Friction.

The value of σ was found to be 0.05. If we draw a line parallel to the line representing the central value at a distance of 3σ in Fig. 30 we get 0.1628 as the lower limit for the value of coefficient of friction. According to probability analysis this means that there is one chance in about 374 that there can occur a value lower than 0.1628 for coefficient of friction.

c. Analysis of all joints with mill scale surfaces omitting 4 joints.

In order to arrive at a more reasonable value, a second analysis was made omitting 4 joints with 2 or 3 bolts having the value of coefficient of friction greater than 0.45. These values were considered too high to deserve the weight given to them by this method of analysis. Moreover we are more concerned with low values than very high ones.

The central value of coefficient of friction for the rest of the 123 joints was 0.3062. The standard deviation was found to be 0.0316 which gave 0.2114 as the value for lower limit for coefficient of friction (Fig. 22).

IV. Conclusions.

The results of the three facets of the presented investigation allow the formulation of several conclusions.

A. The Effect of Misaligned Holes.

The tests of specimens in which the misalignment extended over two rows of bolts did not reveal any significant effect upon ultimate strength or efficiency. The ultimate strength and efficiency of joints of a single row of bolts, in which only one outside hole was misaligned, showed slight decrease of both ultimate load and of efficiency.

B. The Effect of Punched Holes.

The ultimate strength, efficiency, elongation, and strain-energy absorption of the joints with punched holes at room temperature were definitely less than those with drilled holes. Ultimate strength and efficiency were decreased about 10%; elongation and strain-energy absorption, 40%.

C. The Effect of Painted Faying Surfaces.

Red lead paint on the faying surfaces was found to have a very definite effect on the friction developed between the faying surfaces at room temperature. The specimens with painted faying surfaces began slipping at approximately one-third the working load.* The specimens with unpainted mill-scale faying surfaces did not slip until after the working load had been reached. At minus 24 degrees Fahrenheit, however, the red lead painted faying surfaces developed practically the same friction as the unpainted mill scale. However this does not seem to have any noticeable effect upon the efficiency of the joint.

D. The Effect of Low Temperature.

At minus 24 degrees Fahrenheit the joints failed with a brittle fracture. At this temperature the fractures were 100% cleavage with almost negligible plastic deformation. Although the ultimate strength and the efficiency of the

*Working load was taken to be that load which produced an average shear on the gross area of the bolts of 15,000 psi.

joints were decreased only 25%, the elongation and the strain-energy absorbed were decreased about 80%.

Low temperatures did have one beneficial effect on the specimens. Even though the faying surfaces were painted with red lead, the joints did not slip until the working load had been reached.

E. The One-Turn-of-the-Nut Method.

1. The one-turn-of-the-nut method of tightening the bolts reduces the slip in bolted joints appreciably by providing higher clamping forces to the plates.

2. For joints tightened by the one-turn-of-the-nut method, the length of grip of the bolts plays an important part in the slip characteristics of the joints.

3. The nominal coefficient of friction for first major slip seems to be lower in joints tightened by the one-turn-of-the-nut method than in joints tightened by the required minimum bolt tension. Nevertheless due to the higher clamping force the load which will produce slip is also higher.

4. The partition of the load in joints tightened by the one-turn-of-the-nut method is similar to that tightened by the required minimum bolt tension.

5. The efficiency computed by the Schutz-Newmark formula agreed closely with the experimental value in specimens with bolts tightened by the one-turn-of-the-nut method, even though the formula would not strictly apply to the one in which the bolts sheared.

F. The Coefficient of Friction.

1. The most important factor in determining the value of coefficient of friction in a bolted joint is the condition of the faying surfaces.

2. Factors reducing the coefficient of friction are: increasing bolt tension, decreasing tension:shear ratio, increasing size of the joint, increasing number of bolts in the joint and sustained loading.

3. In statistical terms, the lack of control increases with decreasing size of joint and decreasing number of bolts.

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Journal of the
STRUCTURAL DIVISION
Proceedings of the American Society of Civil Engineers

INSTALLATION AND TIGHTENING OF HIGH-STRENGTH BOLTS[a]

E. F. Ball,[1] M. ASCE and J. J. Higgins

SYNOPSIS

This paper describes the procedures which have been used by Bethlehem Steel Company to insure that high-strength bolts are installed and tightened to produce the minimum bolt tension required by specifications. It outlines the studies and tests that have been made to develop a safe, practical, and economical method of bolt tightening. Details of present procedures are described and illustrated.

The use of high-strength steel bolts as permanent fasteners for structural steel joints has become general only since the formation of the Research Council on Riveted and Bolted Structural Joints in 1947, and the publication of specifications approved by the Council in 1951. However, the use of heat-treated carbon bolts, torqued to a high tension, is not new. Bethlehem Steel Company, and probably others, used such bolts for temporary erection purposes in the early thirties, and have continued to use them in erecting all structures of any magnitude since the building of the Golden Gate Bridge.

The determination of the number of heat-treated bolts required in a stress-carrying joint was based on shear values about 50% greater than those permitted for rivets of the same diameter. It is also of interest to note that washers of ordinary carbon steel, not heat-treated, were utilized in the operations, and the nuts used were standard size, not heavy and not heat-treated.

Fig. 1 shows a close-up view of a bridge joint fitted up not only for riveting, but also for carrying erection loads. This is a joint in the New Orleans Bridge across the Mississippi River which is now under construction. The one side of the diagonal member connection shown has 149 holes for fasteners and you will note we have used 50 pins and an equal number of bolts to hold

Note: Discussion open until August 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1974 is part of the copyrighted Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. ST 3, March, 1959.

a. Presented at ASCE Convention in Chicago, Illinois, February 1958.

1. Chief Engr., Fabricated Steel Constr., Bethlehem Steel Co., Bethlehem, Pa.

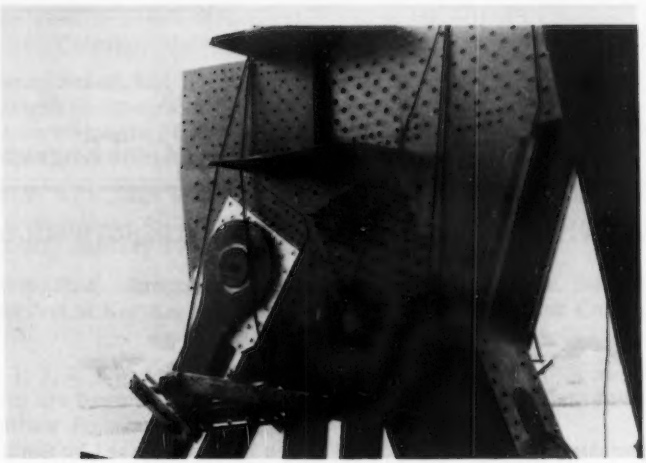

Fig. 1

the joint which, in this case, is sufficient for fitting-up purposes and for carrying load encountered in the course of erection. While this joint will be riveted eventually, it is very similar to one using A-325 bolts as permanent fasteners. The temporary bolts shown are high-strength with regular pattern nuts and mild steel washers.

In the pre-Research Council days no attempt was made to attain any precise amount of minimum tension or clamping force in the bolts. The bolts were tightened with impact wrenches and operators were instructed only to make certain that they were really tight with the joint material in contact.

With the advent of the specifications for using bolts in permanent connections, engineers of course became interested in more precise methods of obtaining the required tension in the bolts. The specifications state the minimum tension necessary to obtain clamping power, but do not require any specific method to be used to attain the tension. Some erectors have used hand torque wrenches alone for tightening. Others tightened with impact wrenches and checked with hand torque wrenches. Bethlehem followed the latter practice. It was soon recognized by various experimenters that the approximate torque values given in the specifications are not reliable measures of the tension in the bolts. The amount of torque required varies with the lubrication used on the bolts, and with small differences in the threads of bolts and nuts. Hand torque wrenches have to be calibrated for each lot of bolts at each application to determine the correct torque-tension ratios for that application.

Bethlehem very quickly came to the conclusion that bolts must be tightened with impact wrenches for reasons of economy and practicability. After tightening to a degree governed by the intuition of the skilled operator, a certain percentage of the bolts were checked with calibrated torque wrenches. Bolts were re-torqued or replaced when checks proved unsatisfactory. This method worked, but it placed a premium on the use of skilled operators who were accustomed to the "feel" of bolts when they had attained proper tension.

The next method developed by Bethlehem was the use of calibrated impact wrenches. This method required the maintenance of a constant air pressure,

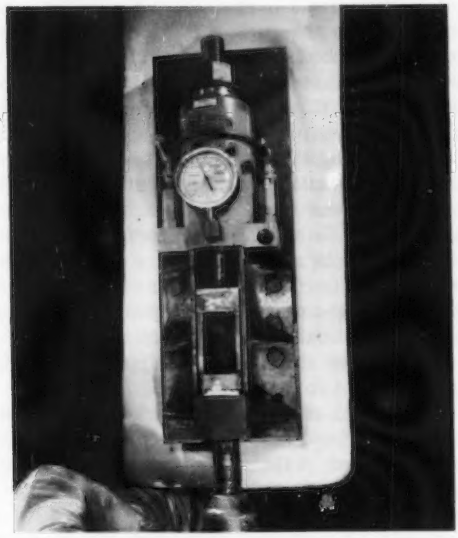

Fig. 2

adjusted to stall the wrench when proper tension was reached. Correct amount of air pressure was determined by testing wrenches periodically on the job with a calibrating device.

We designed and built a calibrating device which is shown in Fig. 2. It consists of a hollow ram hydraulic jack of 60-ton capacity. The yoke which pulls against the ram is guided by a substantial frame which bears against the cylinder, and by torquing a bolt held between the yoke and bottom face plate, the jack is energized by the stress in the bolt. A hydraulic gauge, reading total load, indicates the bolt tension directly. This device was used by Bethlehem and had much merit.

Fig. 3

Later on, a similar device was made and marketed by another firm. It is shown in Fig. 3 and Fig. 4.

The calibrated wrench method was used to tighten several million high-strength bolts all over the country and was successful since fewer skilled men are required to do the work. It proved to be economical and practical when the bolts on a job were mainly of the same diameter and approximately the same length, thus requiring the same air pressure. However, it became quite impractical for joints requiring different size bolts which meant additional wrenches, with different air pressures. One joint in a modern multi-storied building may demand 4 or even 5 sizes of bolts, from 3/4 in. dia. to 1-1/4 in. dia. Each size of bolt needed a separate impact wrench calibrated to stall at some determined air pressure. The complexity of using so many wrenches with so many air pressures at a single joint was increased by the need of having still more air hoses carrying air at the higher pressure necessary to ream out holes, cut out shop rivets, chipping, etc. Manifestly some other method of tightening had to be developed.

We realized, even when our method of calibrating impact wrenches was doing a good job, that something must be done to reduce the amount of equipment needed. We felt one of the most promising developments would be a method of torque control which would permit us to operate at any air pressure, preferably 100 psi or more, to eliminate the need for a separate air line to ream, chip, etc. An adjustable friction clutch device was developed that could be applied to the impact wrench between wrench and socket. The clutch could be adjusted to slip at prescribed torques and would hold its adjustment fairly well. However, it added weight to the power tool and its size prevented use in close quarters, both of which made it undesirable.

The air tool manufacturers also studied the problem and finally developed a built-in torque control which would shut off the air pressure at specified torques. The adjustment or settings still had to be made experimentally and checked daily using a calibrating device. This tool worked fairly well, but in no way helped us with our growing and larger problem of joints having several sizes of bolts, or with another problem which presented itself when we had to tighten bolt heads instead of nuts.

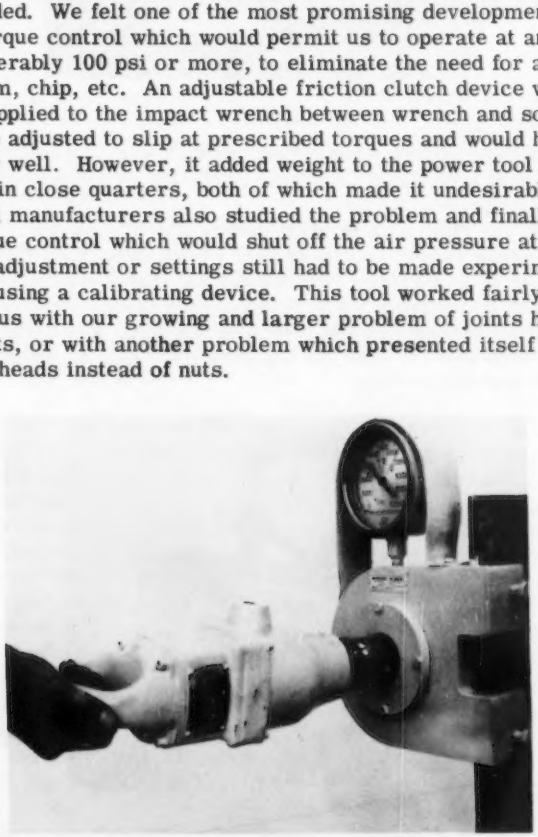

Fig. 4

Since these torque control tools measure torque, and it requires far more torque to properly tighten a bolt from the head than it does from the nut end, it was clear that torque did not provide the measure of tension needed.

It was obvious that we needed a measure of tension other than torque to do a good job, and that we must develop a more foolproof as well as more practical method of bolting.

While Bethlehem, and other firms, were searching for a more practicable method of tightening, the American Association of Railroads was trying to find a good reliable way to tighten bolts at remote locations, without power tools, and using unskilled labor. In 1954 Mr. E. J. Ruble, Research Engineer Structures of A.A.R., conducted a large number of tests to determine if the number of turns of the nut could be used as the criterion for bolt tension. The report of his tests was published in American Railway Engineering Association—Bulletin—Volume 56, No. 520, dated January 1955, and in ASCE Proceedings Paper 786 by F. P. Drew, Assistant Research Engineer Structures of A.A.R.

Mr. Ruble's tests covered bolts ranging from 5/8 in. to 1-1/8 in. in diameter using laminated material drawn up tight with the A-325 bolts used as fitting-up bolts. He found, for all diameters, that 1/2 turn from a "finger tight" position produced the minimum bolt tension required by specifications. He also found that 2 to 3 turns were required to break the bolt or strip the threads. He recommended that bolts be given one full turn of the nut from "finger tightness" in order to be above the minimum tension required. In 1955, the Research Council, in Appendix B to the specifications, approved one turn of the nut from hand-tight position as a satisfactory method of tension control. In all of the above it should be noted that the turns are measured from the finger or hand-tight position after the steel surfaces have been drawn together with fitting-up bolts.

Bethlehem tried this one-turn method but soon found that it was not practical to use finger or hand tightness as a reliable starting point for measuring the amount of turn. Because of the effect of dents and dirt accumulating in threads, experience proved it was difficult and time consuming to determine

Fig. 5

the hand-tight position. Furthermore, steel erectors objected to tightening bolts by hand when they had impact wrenches available to do the work. The need for establishing a better starting point for final stressing was clearly indicated. It was also evident that all tightening should be done with impact wrenches for practical reasons.

After numerous experiments, Bethlehem developed a turn-of-nut method which has proved to be safe, practical and economical. From our experience with riveting we knew that drawing the component material together properly, commonly called fitting-up, must precede any routine procedure for final bolting. Therefore, we have established fitting-up as the first operation to be performed.

Next we had to determine how to obtain a reasonably uniform starting point from which we could measure nut turn. We found it by adapting a procedure workmen normally use in power bolting. We have them install the bolts with the nuts started on a thread or two by hand, then spin the nut down tight, using the impact wrench as a nut runner, to a "snug" condition. The most reliable starting point, for measuring the amount of turn, was found to be this "snug" condition. This condition is obtained by spinning the nut until the wrench starts to impact, which occurs when turning of the nut is resisted by friction between the face of the nut and the surface of the steel. This "snug" position is naturally tighter than the hand-tight position and, therefore, less additional turning is required to bring the bolt to its proper tension. Numerous trial runs have shown that the amount of turn required from the "snug" position varies from 1/2 turn to 3/4 turn depending on the bolt diameter and grip. The table shown in Fig. 6 gives the nut turn required for bolts of the most common diameters and lengths.

This table was determined by trying various amounts of nut-turn and measuring the tension obtained in the bolts. Bolts tightened in accordance with the table will have no less than the minimum tension specified by the Research Council.

It may be of interest to examine Fig. 7, on which we have plotted an actual stress-strain curve for a 1" dia. x 4" long bolt which was tested in our Bethlehem plant laboratory. We have indicated on this curve the tension

Bolt Diam, in.	Bolt Grip, in.	Bolt Length, in.
Torque at least 1/2 turn		
3/4	to 5	to 6-1/2
7/8	to 5	to 6-1/2
1	to 8	to 9-1/2
1-1/8	to 8	to 10
1-1/4	to 8	to 10
Torque at least 3/4 turn		
3/4	above 5 to 10	above 6-1/2 to 12
7/8	above 5 to 10	above 6-1/2 to 12
1	above 8 to 12	above 9-1/2 to 14
1-1/8	above 8 to 12	above 10 to 16
1-1/4	above 8 to 12	above 10 to 16

No bolt will have less than minimum specified tension.

Bolt tensions required are as follows:

Bolt Diam. in.	*Minimum Tension lb.
3/4	25,600
7/8	32,400
1	42,500
1-1/8	50,800
1-1/4	64,500

*Equal to 90 pct of min proof load.

Fig. 6

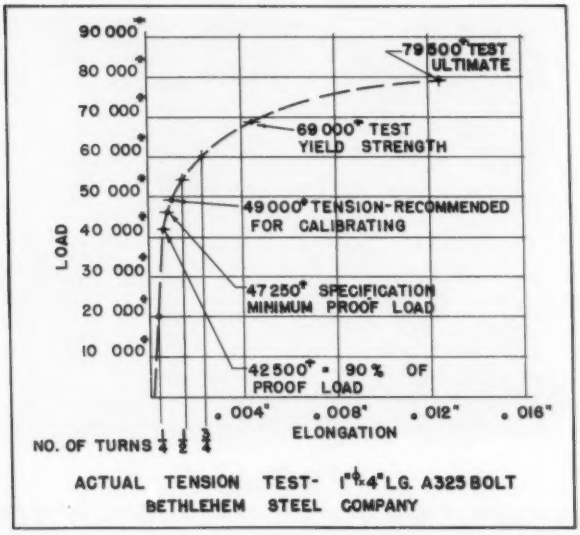

Fig. 7

corresponding to 1/4, 1/2 and 3/4 turn of the nut for a similar bolt from our trial runs. This is an illustration of how our procedure ties in with the specification requirements.

The following instructions for tightening high-strength bolts are given to erectors:

1. Fair-up holes with enough pins to maintain dimensions and plumbness of the structure. Pins are not to be removed until bolts in balance of holes have been tightened.
2. Install bolts in the remaining holes.

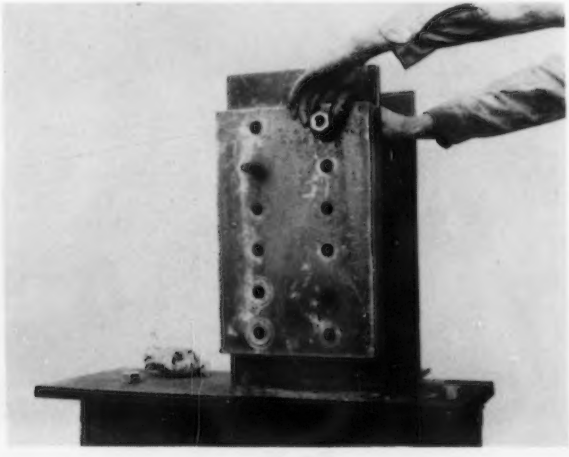

Fig. 8

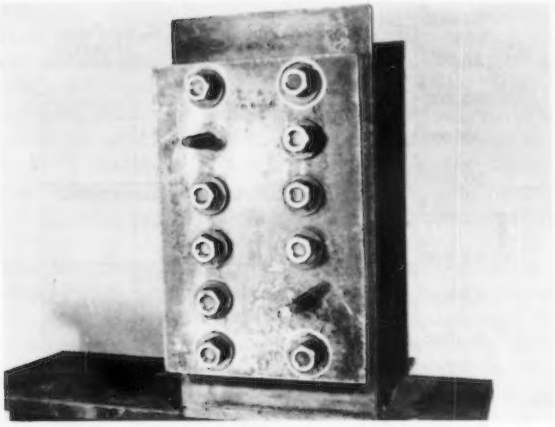

Fig. 9

3. Tighten a pattern of bolts as you would for riveting, being sure that the connected parts are properly fitted.

This instruction deserves comment. It should be emphasized that the steel must be fitted-up so that faying surfaces are in contact before tightening the bolts is started. Some of the permanent bolts already placed in the holes are used for fitting-up purposes, and they are tightened by means of impact wrenches, without regard to the amount of nut-turn. The essential feature in this operation is to make certain that the plies of steel are tightly fit together. The typical truss joint shown in Fig. 11 illustrates the number of pins and the number of fitting-up bolts used in an actual field joint for riveting.

Fig. 12 shows another actual joint to be bolted being fitted-up. You will note the marks indicating which bolts are being used for fitting-up purposes.

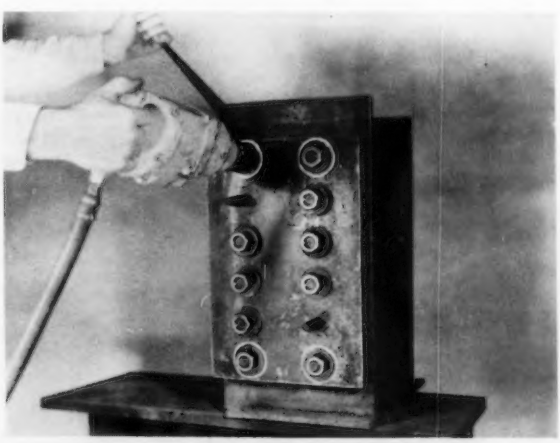

Fig. 10

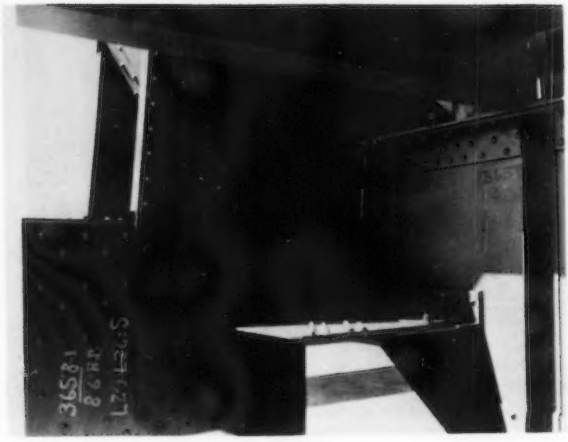

Fig. 11

4. Using an impact wrench, with a marking socket, spin the nut of each bolt not used to fit-up, to a snug condition, then continue to tighten it to the number of turns specified in the table.

This instruction also requires comment. The marking socket referred to as shown in Fig. 14 marks the outside face of the nut or head during the operation, so that workmen or inspectors can see at a glance whether or not the bolt has been tightened. In some cases, for clearance reasons, the head of the bolt must be turned instead of the nut. The same procedure should be followed, with the same amount of turn. Turning the head normally requires a little longer time than turning the nut. The amount of air pressure used for the wrenches need not be constant; it usually varies from 80 psi to 120 psi, depending on type of wrench, diameter of bolt, type of joints, and conditions at the site. Legible marks should be placed on the outside walls of the wrench sockets, 180 degrees apart, for visual determination of nut-turn.

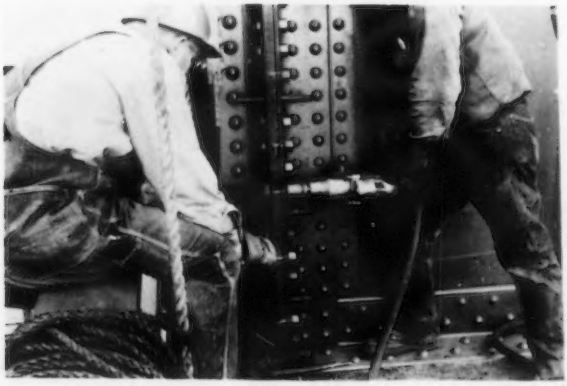

Fig. 12

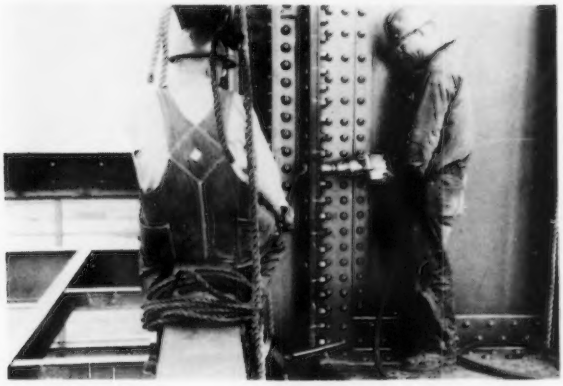

Fig. 13

Marks must be differentiated either by color or by number of marks.

5. Replace pins with bolts and tighten as in item 4.
6. Check the tightness of each fit-up bolt with impact wrench. Usually no further torquing is required, but some may require 1/4 turn.

Fig. 17 shows the completed job and clearly shows the marks of the tightening socket on the face of the nuts.

7. Tightening crew should mark completed joint with identifying symbol. Fig. 18 shows a typical completed joint before painting.

All bolted joints should be inspected by the contractor, and by the owner if he desires. Inspectors should become thoroughly familiar with the bolting procedure and insist that it be followed in accordance with the instructions.

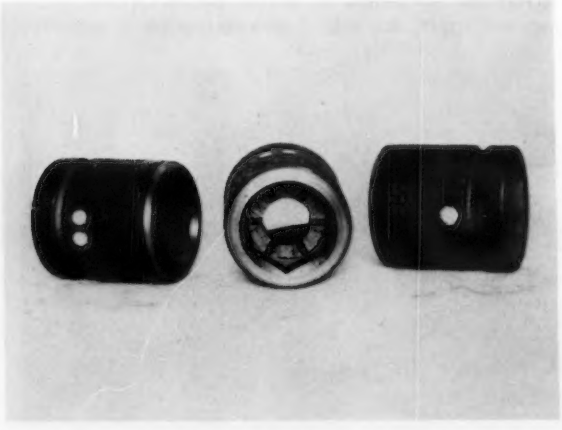

Fig. 14

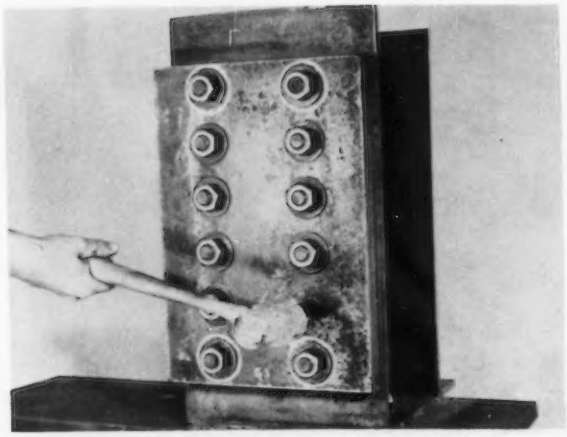

Fig. 15

As joints are completed, inspector should note that they bear identification symbols. He should see that all nuts have a mark on the outside face, made by the wrench sockets during tightening. He should also check a number of the bolts mechanically, as shown in Fig. 19, to see that they have been torqued to at least the specified minimum tension.

This check can be made with a hand torque wrench, with a dial gauge calibrated on the job to determine the torque-tension ratio. Fig. 20 shows a hand torque wrench being calibrated by means of a hydraulic calibrating device.

A heavy mill-type building is pictured in Fig. 21.

Building shown is the secondary crusher unit for Erie Mining Company at Aurora, Minn. It contains 4,000 tons of steel and 70,000 A-325 fasteners. It

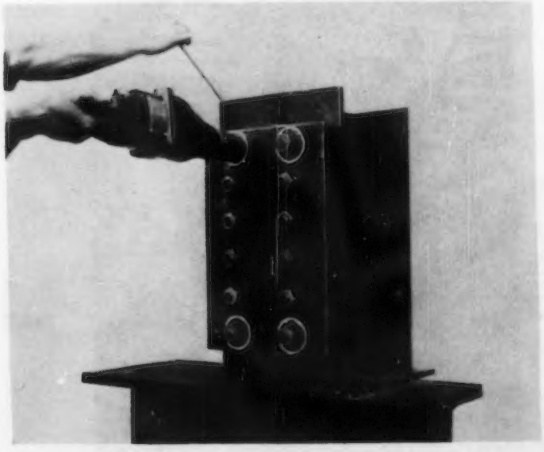

Fig. 16

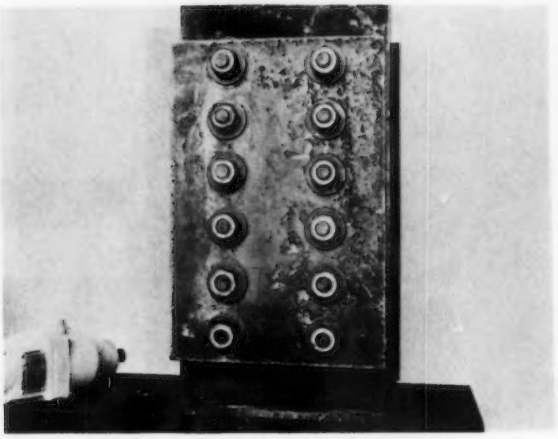

Fig. 17

is a tall structure which required cranes with 140' booms plus 30' jibs to erect it.

A typical highway bridge structure is shown in Fig. 22.

This shows two adjacent water crossings on the northeast extension of the Penna. Turnpike. The continuous girders span the Lehigh River and the deck trusses are over a creek. 75,000 A-325 bolts were used to fasten 4,200 tons of material.

A modern New York skyscraper is shown in Fig. 23. This is a 38-story building at 666 Fifth Avenue, New York City. The building is over 500' in height, requiring 13,000 tons of structural steel and 240,000 A-325 bolts.

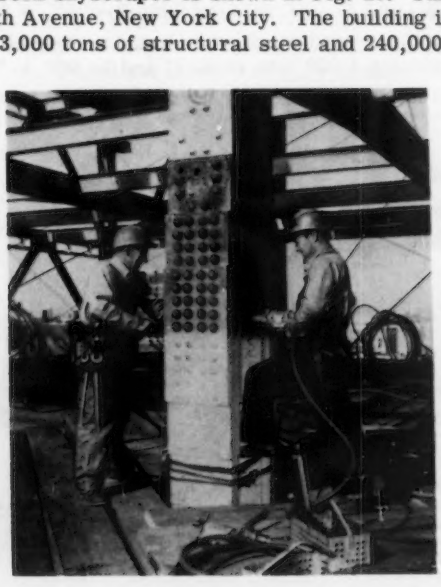

Fig. 18

Fig. 19

The last three slides are representative of the type structures which are now being bolted. They were selected from many examples of comparable size and importance.

All of these structures utilized high-strength bolts for field connections, and all bolts were tightened in accordance with the procedures described above.

Since 1951 Bethlehem's erection department has installed and tightened over 15,000,000 high-strength bolts, by various methods, with no record of failure. The present method has proved to be reliable, practical, and economical. However, Bethlehem and others are still searching for improvements. Equipment manufacturers are always improving their impact wrenches and accessory equipment and trying to develop new and better types. It is,

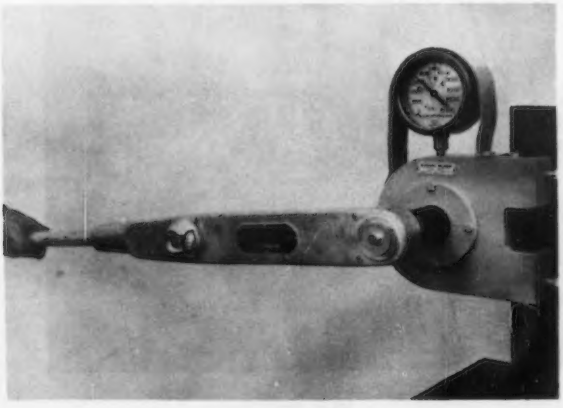

Fig. 20

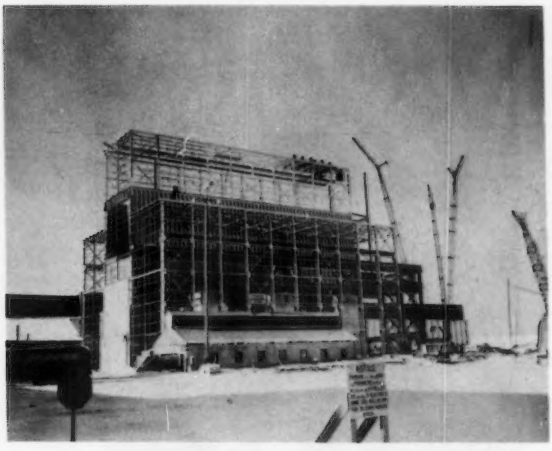

Fig. 21

therefore, quite possible that the method of tightening described in this paper, even though it is satisfactory, may be superseded by some better method in the future.

Fig. 22

Fig. 23

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Journal of the
STRUCTURAL DIVISION
Proceedings of the American Society of Civil Engineers

EXPERIENCE WITH HIGH-STRENGTH BOLTS IN THE MACKINAC BRIDGE[a]

John W. Kinney[1]

ABSTRACT

A brief description of the project is given, and details are described as to the particular application of bolted connections. The procedure for bolting and inspection as developed on the project is outlined. Some of the more common troubles are described and illustrated. Conclusions resulting from experience on the project are drawn.

The use of more than 1,000,000 high-strength bolts in the Mackinac Bridge was an important factor in keeping steel erection on schedule and allowing opening of the bridge to traffic on the scheduled date, November 1, 1957.

A brief description of the Mackinac Bridge will include many outstanding features. Its cost, $100,000,000, including administration, engineering, and interest during construction, makes it the most costly bridge project yet undertaken. Its length, over the often rough waters of the Straits of Mackinac, of 17,917 feet, or over three miles, rates the bridge as a major structure. The center suspended span of 3,800 feet ranks second only to the Golden Gate's center span of 4,200 feet. The two side spans of 1,800 feet each, two unloaded backstay spans of 472 feet each, and the two anchorages of 135 feet each, make the total length of the suspended span structure 8,614 feet from back to back of anchorages, the longest single suspension bridge yet built.

In order to make financing possible, it was essential that the bridge be completed to the point of opening to traffic in four construction seasons from the time of financing. Winter conditions at the Straits of Mackinac make marine operation impossible during the winter months. The construction

Note: Discussion open until August 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1975 is part of the copyrighted Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. ST 3, March, 1959.

a. Presented at the February 1958 ASCE Convention in Chicago, Ill.

1. Resident Engr. for D. B. Steinman, Cons. Engr., N. Y., N. Y.

season is short, usually from about the middle of April to the middle of December for marine work. Interruptions to the work during the construction season were caused at intervals by storms, or by winds and rough seas of less than storm intensity.

The contract for the superstructure steel, awarded to the American Bridge Division of U. S. Steel Corporation, was a unit price contract for $44,532,000, the largest bridge contract awarded to date.

The 1954 season was devoted to foundation work on the main bridge piers, and the start of steel fabrication on the main towers.

Steel erection was started on July 2, 1955 on the main towers. The towers were topped out to their height of 552 feet on December 6. The backstay spans were floated out in November and December. This schedule allowed the start of cable work early the following season and its completion in 1956. Riveting of the towers and backstay spans proceeded during their erection and assembly. It proved difficult to maintain scheduled progress on riveting, due to a lack of qualified men. Consequently, the towers went through their first winter with the riveting approximately 60% complete from the bottom up. The upper unriveted splices were bolted and pinned in 40-60 ratio. Similarly, the backstay spans were not completely riveted, and some of the chord splices and other connections went through the winter bolted and pinned.

The difficulty of maintaining the riveting schedule led the American Bridge Division to propose the use of high-strength bolts for all of the approach truss spans scheduled for 1956, and the suspended spans scheduled for 1957. This proposal was accepted by the consulting engineer. Somewhat over 1,000,000 high-strength bolts were required for this work.

All of the steel to be bolted had been designed for 7/8 inch and 1 inch rivets. All of the truss chords, both in the approach spans and in the stiffening trusses for the suspended spans, were assembled and reamed before erection.

The truss approaches, totaling 9,303 feet in length, consist of seven continuous span units of four spans, the longest unit being 480-560-560-480 feet in length. The depth of the trusses in these long spans is 45 feet at the center of the span and 53 feet over the intermediate piers. Construction is of the deck truss type, with the floor beams, 54 feet long, being framed through the trusses which are on 34 feet centers. Silicon steel, A.S.T.M. A94, was used for highly stressed truss chords and diagonals, and structural carbon steel, A.S.T.M. A7, in the balance. The maximum grip of bolts used in the connections is 4-1/2 inches, and the number of plies connected is up to six. Stringers were spliced to develop continuity over several floor beams. Erection of all members except the floor system was done from the water by the cantilever method, by the means of anchoring adjacent trusses, and the use of falsework of which there was usually one bent in each span, and full cantilever erection in the fourth span of each unit.

The suspended spans are of the deck truss type, with stiffening trusses, 3,766 feet long in the center span, and 1,775 feet long in each side span. The depth of the trusses is 38 feet, and the panel lengths about 39 feet. Trusses are 68 feet on centers, and are provided with a top and bottom lateral bracing system as well as sway bracing. Floor beams, 68 feet long, are of truss construction, and were shop assembled and riveted. Silicon steel was used in the stringers, stiffening truss chords and other highly stressed members. The maximum grip in the bolted connections is 5 inches, and the maximum number

of plies connected is six. Units of 2 panel lengths were assembled and bolted with high-strength bolts in an assembly yard on shore.

The assembled units consisted of both trusses, the connecting floor beams, and top and bottom lateral bracing. The units weighed from 100 to 150 tons, and were barged from the assembly yard to the site, two units per barge. The units were raised by means of lifting struts which were rolled on top of the main cables. The two lifting struts used for each lift were each provided with two 9 part falls, which were attached to the floor trusses of the unit. When the units were lifted into place, the four suspender ropes, already installed, were connected to their brackets on the trusses by means of socketed connections on the suspenders. Balanced erection both ways from each tower was followed. A feature of stiffening truss erection is the fact that the final shape of the truss is not developed until all dead load is applied. This makes it impossible to make up the chord and diagonal connections between the units as erection proceeds. As the stage is reached where the joints make up and the holes are fair, which may be a temporary condition, the connections must be bolted and pinned in the case of riveted connections. A big advantage in the use of high-strength bolts in these connections is that bolting can be permanently made at scattered points as soon as they are fair and ready, without the necessity of temporary bolting and pinning. Final bolting of the chords and diagonals was kept well ahead of second pass erection.

The erection of the floor system followed as a second pass. All of the stringers, cross beams, steel grid flooring, and railing, comprising the second pass, were hoisted at the towers and erected in a balanced sequence from the towers. Stringers and cross beams were bolted 100 per cent as erected. This was immediately followed by the placing of the floor grid, which was then welded sufficiently to carry the erecting crane and straddle buggy. All material for the second pass was moved from the towers to the point of erection over the grating by straddle buggies; likewise the truck crane shuttled between erection points on opposite sides of each tower. The use of high-strength bolts speeded up the placing of the grating and the use of the floor system.

Some examples of bolted connections are shown in Fig. 1, Typical Bottom Chord Splice in Approach Trusses; Fig. 2, Typical Bolted Splices at Top Chord of Stiffening Truss; Fig. 3, Connection to Cable at the Center of Stiffening Truss.

Before the start of bolting, the Specifications for Assembly of Structural Joints Using High-Strength Steel Bolts, approved by the Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation of February, 1954, were agreed on.

Before the use of high-strength bolts was agreed on, early in 1956, some of the approach steel had already been shipped to the job. The specifications (for riveted joints) called for a coat of lacquer on the contact surfaces, one coat of red lead and oil paint on the balance. The lacquer coat as well as the red lead coated surface under the washers of the bolts was removed by sandblasting.

One of the first questions to come up in the field checking of bolts was the possibility that the bolts could be overtightened, as well as undertightened. As the truss chord connections, and others, are subject to reversal of load, and because of the importance of the bolted joints, it was agreed that everything possible would be done to insure against possible damage due to overtightening, as well as the possibility of undertightening.

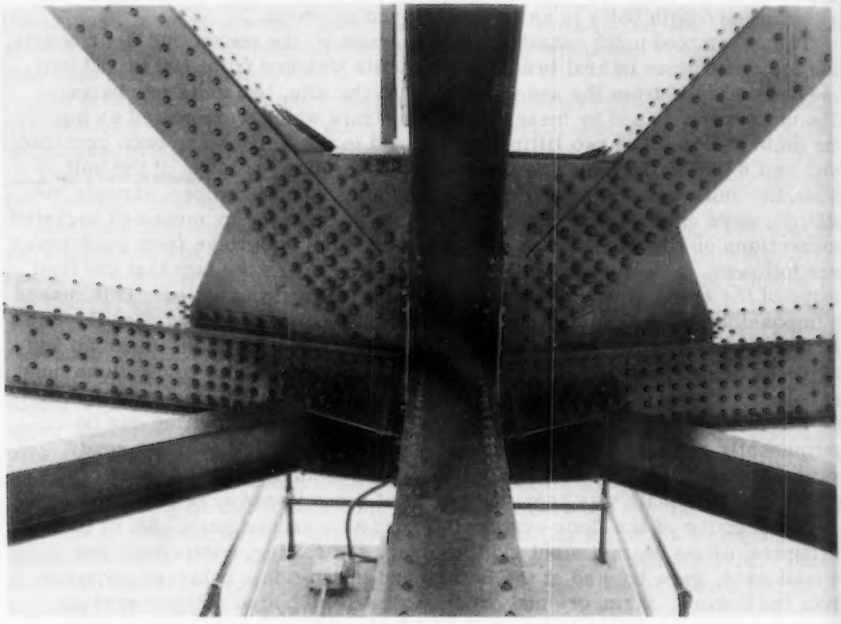


Fig. 1: Typical Bottom Chord Splice in Approach Trusses.

Test runs were made on a number of bolts in a Skidmore-Wilhelm calibrator. The 7/8" bolts used in this series were tightened to the required tension. Air pressure was adjusted by means of air line pressure regulators so that the required bolt tension was reached in 10 seconds. Torque readings were then made on each bolt for comparison with the tabular value. Chicago Pneumatic Model CP 610 impact wrenches were used. A set of new wrenches was procured by the contractor for use on high-strength bolts to insure uniformity of equipment. A number of tests on new 7/8" bolts showed torque readings close to 400 lb. ft. at the recommended bolt tension for calibrating of 37,000 lbs. Similar tests on new 1" bolts showed torque readings close to 700 lb. ft. at a bolt tension of 49,000 lbs. Tabular values are 480 lb. ft. and 710 lb. ft. for non-lubricated bolts and nuts. The observed difference in torque was considered due to the light oil film present on the nuts as furnished.

With a stress area of 0.4612 sq. in on 7/8" bolts, and 0.6051 sq. in. on 1" bolts, the tension of 37,000 lbs. on 7/8" bolts is equal to a stress of 80,225 p.s.i. and the tension of 49,000 lbs. on 1" bolts is equal to a stress of 80,978 p.s.i.

It was found in the early stages of bolting that over 10% of the bolts in the connections torqued over 600 lb. ft. in the case of 7/8" bolts. If the torque in the calibrator at 37,000 lbs. tension was correct at 400 lb. ft., a torque of 600 lb. ft. would indicate that the bolt was stressed to about 120,000 p.s.i. or over the minimum ultimate strength of 115,000 p.s.i. It is probable that the lubrication failed, giving higher torque readings, before such stress was reached.

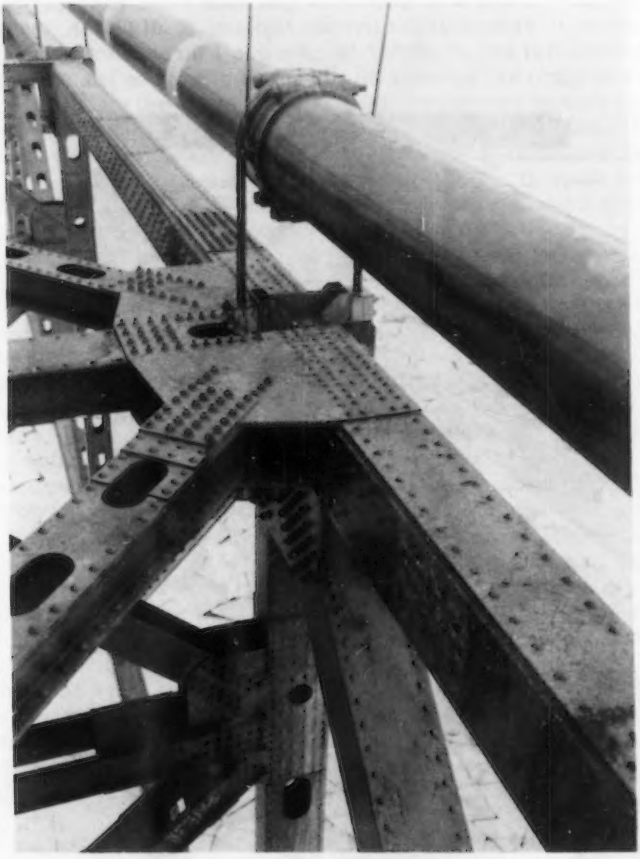


Fig. 2: Typical Bolted Splices at Top Chord of Stiffening Truss.

A number of tests by tightening to failure with an impact wrench showed that the nuts would eventually strip. In no case did we find failure by breaking the bolts, or by stripping the bolt threads.

In order to observe the condition of the overtorqued bolts, a number of 7/8" bolts were removed, with torques ranging up to 600 lb. ft. The bolts were checked with a thread gauge for stretch or buttressing of threads and the nuts were checked for excessive looseness. No apparent damage was observed. A second series of 18 - 7/8" bolts torquing over 600 lb. ft. was removed for examination. This group appeared representative of bolts above 600 lb. ft. (Table I). Torques ranged from 640 lb. ft. to 780 lb. ft. At 640 lb. ft. some galling, and some polishing of the bolt threads was noted on some of the bolts. Some binding of either a new or an old nut was noted. At 680 lb. ft. slight damage to the crest of the bolt thread was noted on one bolt. In most cases there was some restraint in running a new or the original nut on the bolt. At 720 lb. ft. slightly increased damage to the crests of bolt threads was

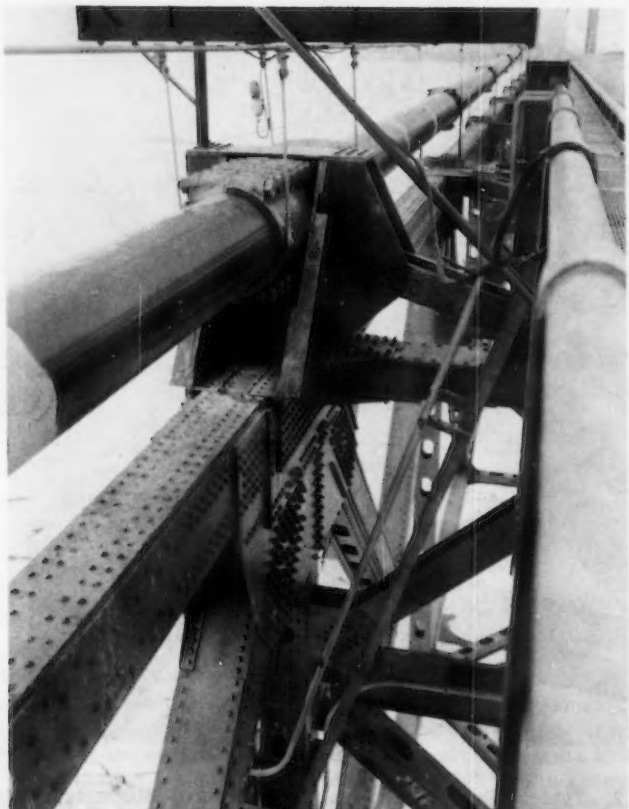


Fig. 3: Connection to Cable at the Center of Stiffening Truss. The Pin Connection is not Shown.

noted, and generally more difficulty in running on a new or original nut was encountered. At 740 lb. ft. one bolt showed slight damage to crests of threads and difficulty in running on a new nut. At 780 lb. ft. one bolt showed no apparent damage and no trouble in running on a new or the original nut.

It appeared from the above that 600 lb. ft. torque was as high as could consistently be allowed without probability of damaging a 7/8" bolt.

Since the ultimate failure by tightening appeared to be caused by stripping the nut, the contractor proposed to salvage any 7/8" bolts which torqued above 600 lb. ft. as long as there was no visible damage, and a new cut could be run on the bolt without restraint.

Several objections to this course are apparent. Even though there is no visible damage, there is a possibility (1) that the bolt threads may be slightly buttressed, causing loss of contact area and excessive contact pressure on the threads of nut and bolt, (2) if the bolt has been stretched beyond its elastic limit, the thread pitch is changed, resulting in excessive thread pressures and cumulative failure, (3) polishing, or the opposite action of galling the bolt threads, will throw the torque-tension relation completely off, or (4) the character of the bolt steel may have been changed by work hardening of the threads, with possible embrittlement, which might lead to failure.

In an effort to get more information on the overtorqued bolts, it was decided to break some of the above samples in a testing machine; and Professor Robert Brown of the Sault Branch of Michigan College of Mining and Technology was commissioned to do this. The results are shown in Table I.

Two new 7/8" bolts were picked as a control. There was a considerable variation in the ultimate load, but both bolts broke in a normal manner through the thread, see Plate 1, bolts A and B. The maximum stresses were 104,900 and 130,500 p.s.i.

Six bolts on which the torque reading was from 640 to 780 lb. ft. and on which a new nut could be run without restraint, were selected for testing with new nuts. All of these bolts failed through stripping of the nut thread. The ultimate stress was quite variable, as expected, and in some cases was surprisingly high, 139,000 p.s.i. The lowest stress causing failure was 71,500 p.s.i.

The engineers concluded from the above that all 7/8" bolts which torqued above 600 lb. ft. should be removed and rejected, and that torque exceeding 1,050 lb. ft. be cause for rejection of 1" bolts. In both cases, the figure is 150% of the torque producing the recommended tension for calibrating.

The procedure was set up to torque all bolts. Any bolts torquing below 400 lb. ft. for 7/8" bolts, and 700 lb. ft. for 1" bolts were tightened further by a partial turn of the nut with the torque wrench at the time of torquing. In some cases, it was found that the torque would not come up on tightening, indicating that failure had started, and such bolts were rejected. The nuts were not backed off before reading the torque, as non reversible torque wrenches were used.

The tightening procedure outlined above was followed during 1956. Rejections came down to an estimated 3 to 4%.

The turn of the nut method of tightening was tried, but consistent results were not obtained, possibly due to the large variation in number of plies, thickness of metal, stiffness of parts, and fit of the connections.

In 1957, the principal bolting remaining was in the suspended spans. A considerable amount of this was done on shore, where closer attention could

TABLE I

TESTS ON HIGH-STRENGTH BOLTS

At Sault Branch of Michigan College of Mining and Technology - May 22, 1956

Overtorqued 7/8" High-Strength Bolts Removed from Truss Connections

No.	Torque Lb.Ft.	Old Nut Run-up *	New Nut Run-up *	Description	Ult. Load Lbs.	Ult. Stress on .4612 sq. in.-p.s.i.
1	720	No	No	Old nut has slight bind		
2	670	No	Yes	No visible damage		
3	720	Yes	No	No visible damage	60,750	131,800
4	640	Yes	Yes	No visible damage	59,650	129,000
5	780	Yes	Yes	No visible damage	61,740	134,000
6	680	Yes	No	No visible damage	32,980	71,500
7	680	No	No	1st thread hit, damaged		
8	680	Yes	No	No visible damage	38,350	83,000
9	680	No	No	Crest of threads damaged		
10	720	No	No	Crest of threads damaged		
11	740	No	Yes	Crest of threads damaged		
12	720	No	No	Crest of threads damaged		
13	720	No	Yes	No visible damage		
14	720	No	No	1st thread hit, damaged		
15	680	Yes	Yes	No visible damage	64,170	139,000
16	640	No	No	Top of threads polished		
17	640	No	No	1/3 of 1st thread galled		
18	640	No	Yes	No visible damage		
A	New Bolt				48,370	104,900
B	New Bolt				60,150	130,500

Loading Rate A, 1, 6, 8 Crosshead, no load, 1" per min.
Loading Rate 4, 15, 5, 3, B Crosshead, no load, 1/10" per min.

Failure: New bolts, normal failure by breaking through thread.
Overtorqued bolts, with new nuts, all failed by stripping of nut.

Specification (7/8"): Ultimate Load - 53,000 lbs.
Ultimate Stress - 115,000 p.s.i.
Stress Area - 0.4612 sq. in.

Accepted Torque: 400-600 Lb. Ft.

Hardness Tests on Bolts and Nuts Selected for Test

Bolt No.	Bolt - Hardness	Nut - Hardness
A - New Bolt	Rockwell C25 - Brinnell 253	
6	" C24 - " 247	
4	" C22 - " 237	Rockwell B76 - Brinnell 139
15	" C24 - " 247	
5		Rockwell B75 - Brinnell 137
3		" B74 - " 135

* Columns 3 and 4 refer to whether or not the old nut, or a new nut, can be run onto the overtorqued bolt by hand.

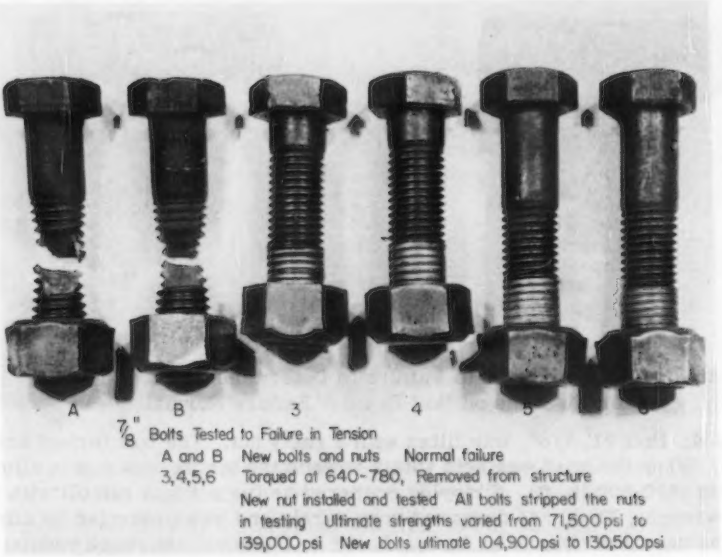

A
B
3
4
5
6
7/8" Bolts Tested to Failure in Tension
A and B New bolts and nuts Normal failure
3,4,5,6 Torqued at 640-780, Removed from structure
New nut installed and tested All bolts stripped the nuts in testing Ultimate strengths varied from 71,500 psi to 139,000 psi New bolts ultimate 104,900 psi to 130,500 psi

Plate 1: 7/8" Bolts Tested to Failure in Tension.

be paid to the bolting, both by the contractor and inspectors. A trial was made of bolting with uncontrolled wrenches, depending mainly on the feel of the wrench. This proved to be satisfactory and to give about the same results percentage-wise as tightening with calibrated and controlled wrenches. Approval of bolting with uncontrolled wrenches was given in order to expedite the bolting. Torquing of all bolts was continued, with the exception of the bolts in the floor system which in general were fasteners only and were not used in shear connections.

It was hoped that torquing could be considerably reduced in line with reported experience on other jobs. While the percentage of cut-outs was low, the occurrence was not uniform. Torquing was continued, both to detect overtorqued bolts and to make the final tightening on undertorqued bolts.

A few other difficulties, for the most part rarely occurring, will be described. Plate 2 shows two 1" bolts in which a fissure developed in the bolt head in cold forming. These bolts broke normally in a tension machine, and apparently the fissures did not affect the strength of the bolt. Such bolts were subsequently accepted.

Plate 3, all 1" bolts: Bolt #1 torqued at 650 lb. ft. On tightening 1/2 turn additional with the torque wrench the torque did not increase. On removal, the bolt was found stretched, which is apparent. Bolts #2 and 3 torqued over 1,050 lb. ft. On removal, they were found to have badly buttressed threads. Further damage to the threads was caused by running the nut off. Bolt # 4 was removed from a misaligned hole. The threads were damaged by being pulled through the hole and the bolt would not develop the required torque even when the head was held while torquing. Further thread damage was caused by removing the nut. This case occurred very rarely.

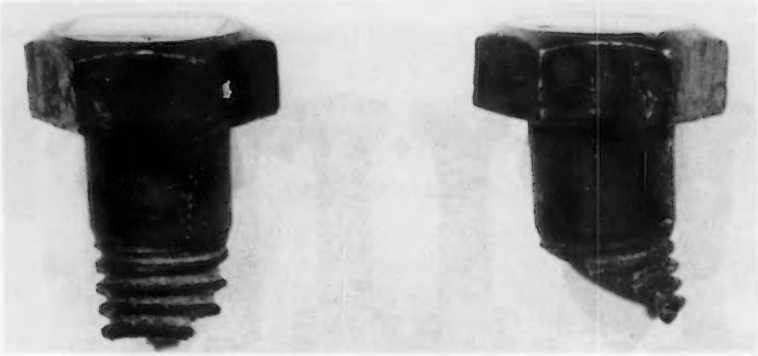
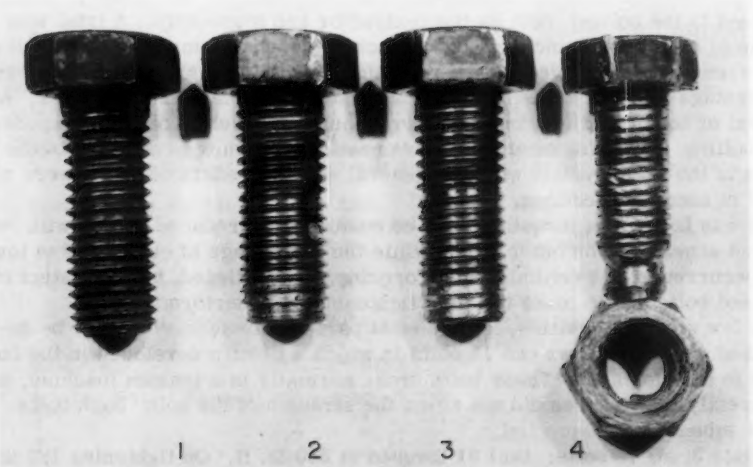


Plate 2: 1" Bolts Tested to Failure to Determine Effect of Piping or Fissures on Bolt Heads. Failure Normal

Plate 4: Bolt #1, 7/8", was fitted with a rusty nut. The bolt turned when torqued. When the head was held with a wrench the torque was within allowable limits (400-600 lb. ft). Stripping occurred as the nut was run off with the impact wrench. This case occurred very rarely and was prevented by eliminating the use of rusty nuts or bolts. Bolt # 2, 1", would not reach required

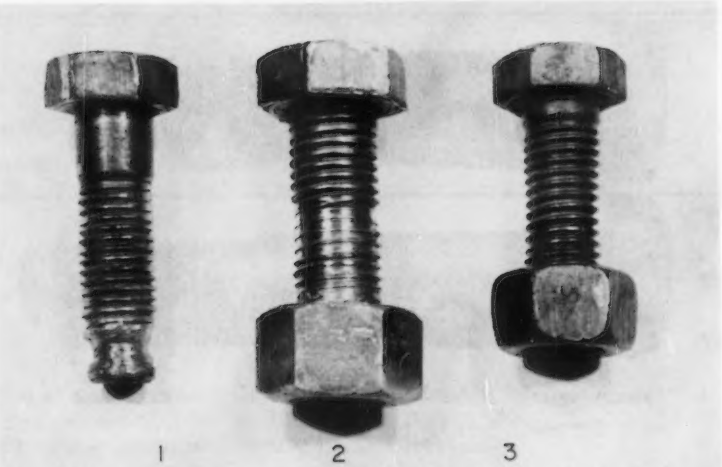


1 Torqued @ 650 Torque did not increase with 1/2 turn additional. Bolt found stretched.

2-3 Bolts found overtorqued badly buttressed threads.

4 Bolt was sucked through a misaligned hole by drawing up on the nut. Bolt turned during torquing but would not develop required torque when bolt was held with wrench.

Plate 3: Illustration of Rejected Bolts.



1 2 3

1 A rusty nut was used on this bolt. Bolt turned when torqued. When the head was held with a wrench the torque was within allowable limits. Stripping occurred as the nut was run off.

2 Would not reach required torque with torque wrench, probably overstressed on original tightening. Impact wrench was reapplied, completing stripping of threads.

3 Bolt torqued at 800 Specified limit 400-600 Note stretch

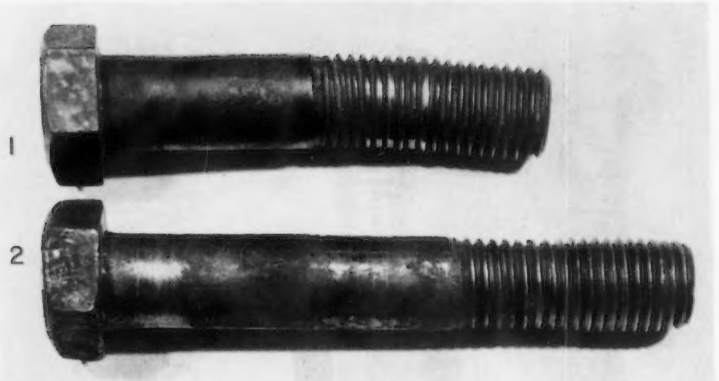
Plate 4: Illustration of Rejected Bolts.

torque with a torque wrench, indicating that the bolt was already overstressed. An impact wrench was re-applied completing the stripping of threads. Bolt # 3, 7/8", torqued at 800 lb. ft. and was removed. Stretch and increased pitch can be noted.

Plate 5, both 1" bolts: Bolt # 1 extended through a structural channel requiring a bevel washer. The bevel washer turned under the nut during tightening, causing the bolt to bend. This effect can be found by observation of the bolt in the connection. Bolt #2 was a soft bolt, included with a shipment of high-strength bolts. This is thought to be the result of an error in the warehouse. No high-strength markings were found on the bolt head. This bolt would probably have passed, except that torquing disclosed that the bolt would not develop the required torque. A further check of the bolt stock disclosed a keg of soft bolts which were discarded.

The instances of failures illustrate the need for thorough inspection by the engineer and careful attention to workmanship on the part of the contractor in order to produce a satisfactory bolted job. It should be pointed out that rejections were rare, probably much less than with a carefully inspected riveted job; and that no instances have been found of bolt failures after the points were corrected and passed.

The following conclusions have been formed by the writer: Considerably more time must be spent on bolt inspection than on rivet inspection both by the engineer and the contractor's forces. There is a definite need for setting an upper limit on allowable torque as well as a lower limit. It has been



1 Hillside washer under head turned while tightening, causing bend

2 Soft Bolt Included with shipment of H. S. Bolts Note stretch Discovered when the bolt would not reach required torque

Plate 5: Illustration of Rejected Bolts.

established that there is a definite possibility of damaging bolts by over-torquing. The trend toward bolting will continue in view of the excellent over-all results, and the better production possible with bolts, also tools and methods will be improved so that inspection time can be reduced.

There is reason for confidence that continued inspection of the Mackinac Bridge will prove that high-strength bolting is completely satisfactory, as well as an expeditious method of making connections on heavy truss work including the stiffening trusses of suspension bridges.

Journal of the

STRUCTURAL DIVISION

Proceedings of the American Society of Civil Engineers

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*There will be no closure.

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APPLICATION OF AASHO SPECIFICATIONS TO BRIDGE DESIGN[a]

Closure by Eric L. Erickson and Neil Van Eenam

ERIC L. ERICKSON[1] and NEIL VAN EENAM.[2]—The discussion has added greatly to the value of this paper and the authors wish to take this opportunity to thank those who have contributed in this manner.

Mr. Archibald has supplied the historical background of the axle loads used in the design of floor slabs. This information is greatly appreciated. With regard to Article 3.6.5, we realize the necessity of making provision for reversals of stress which may occur in members when they are subjected to live load overstresses. The present article provides that when the live load increased by 50 per cent produces live load stresses greater than and of opposite sign to the dead load stresses, counters or compression members shall be used. But the article does not specify the unit stresses which will be permitted in the design of the counters or compression members. That is our principal criticism. It seems that an increase of 25 per cent in the allowable unit stresses should be satisfactory in this case. In regard to Article 3.6.9, the wisdom of one provision was questioned. Under this provision, the area of any portion of a compression member may be neglected in determining the radius of gyration and the effective area. The sole purpose of Article 3.6.9 is to insure that compression members will have sufficient rigidity to prevent objectionable vibrations from the live load and from other vibratory forces. Maximum permissible values of the slenderness ratio L/r are specified which are considered adequate to meet this requirement. The provision which we questioned is used to defeat the purpose. It is always possible to compute a value of the radius of gyration greater than the real value by neglecting areas of the cross section which lie close to the axis. These values are only fictitious, however, and they have no basis in fact. It should not be permissible to use fictitious values in order to justify the use of sections which in reality fail to meet the requirements.

Professor Sawyer has called attention to an error in the "Formulas for Steel Columns" of Appendix B. This error was called to the attention of the Committee on Bridges and Structures of the AASHO and it has been corrected in the Seventh Edition (1957) of the General Specifications.

Professor Wright has called attention to two important matters. The first is the use of a "balanced design" in which fairly high unit stresses are permitted for dead load and relatively low unit stresses for live load and impact.

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a. Proc. Paper 1320, July, 1957, by Eric L. Ericson and Neil Van Eenam.

1. Chf., Bridges and Tunnels Div., Bureau of Public Roads, U. S. Dept. of Commerce, Washington, D. C.
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The Committee on Bridges and Structures of the AASHO has been giving consideration to the adoption of a "balanced design" procedure for some time, but no definite recommendations have been made. As Professor Wright points out, the treatment of design live loads in the AASHO Specifications is unnecessarily complicated. A real effort is being made to study and to simplify Article 3.2.5—Highway Loadings, and it is expected that progress will shortly be made along these lines.

Messrs. Fiore and Kuesel have raised several points. The first concerns the application of the impact allowance. Article 3.2.12—Impact, establishes two classifications, Group A and Group B. Impact is to be added to the items of Group A, but not to those of Group B. The top struts of concrete framed piers and also the columns definitely belong in Group A. The footings, on the other hand, fall in Group B. Following the impact formula, instructions are given for the purpose of promoting uniformity in defining the loaded length "L". These will frequently be found useful.

In regard to Section 5, the AASHO Specifications cover unit stresses in carbon steel, silicon steel, nickel steel and the ASTM A-242 steels. Since silicon steel and nickel steel are no longer used to any great extent in bridge construction, Messrs. Fiore and Kuesel seem to imply that unit stresses and other matters pertaining to design of silicon steel and nickel steel bridges might well be deleted from the specifications. Some States, however, continue to use a limited amount of silicon steel. There are also numerous silicon steel bridges and some of nickel steel, which are rendering excellent service. These must be regularly inspected and rated according to Section 12, Articles 3.12.1 to 3.12.13, inclusive. It would therefore be unwise to omit this information from the specifications at the present time. It is true that T-1 and other special alloy steels have recently been used in particular structures, but since they have not been generally adopted, they do not as yet warrant inclusion. We agree that a general discussion of the factors to be considered in the use of alloy steels would be a valuable addition. A discussion of live loads for long-span bridges is, however, beyond the scope of this paper.

Referring to Section 6—Structural Steel Design, the authors agree that the treatment of long columns and of eccentrically loaded columns in Appendix B is not as complete as it should be. The suggestions submitted by Messrs. Fiore and Kuesel are therefore appreciated.

In regard to Article 3.6.75, the specifications limit the permissible intensity of diagonal tension in web plates. In some cases, this determines their thickness. However, the ratios of thickness to depth given in Article 3.6.75 are concerned primarily with the compression side of the web plate. They were derived by the theory of elastic stability as developed by Timoshenko(11) and by Bleich,(13) and they insure that the web plates will have stability to resist buckling from compression due to bending. When maximum bending and maximum shear occur simultaneously, as they do near the intermediate supports of continuous girders and near the supports of cantilevers, the determination of the necessary web plate thickness to resist buckling becomes very laborious, involving long and tedious cut-and-try methods. In order to avoid the necessity of making some computations, the AASHO Specifications make a reasonable allowance for the effect of the shearing stresses. This was done only after the problem had been thoroughly studied, and the allowance certainly is not an arbitrary one.

In regard to the spacing of transverse intermediate stiffeners, the AASHO formula of Article 3.6.80 is a rather rough approximation of the theoretical treatment developed by Timoshenko(11) and by Bleich.(13) It has the advantage of being simple and easily applied. The formula developed by Hovey in the March 12, 1931, issue of Engineering News-Record is a closer approximation and is superior in that respect. The AASHO formula is based on a safety factor of 1.83 and it also makes a small allowance for the fact that the actual shearing stresses in the webs of plate girders always exceed the average shearing stresses. In spite of their differences, it is believed that if a safety factor of 1.83 is used in Hovey's formula, the two will yield results in close agreement.

Messrs. Fiore and Kuesel are correct in their statement that welding of alloy steels is not to be undertaken without special consideration of the electrodes and techniques required for the specific alloys and conditions to be encountered. The suitability of the chemical composition for the welding process, under the given conditions, should be based upon evidence acceptable to the purchaser.

Regarding Section 9—Composite Action, there are differences of opinion as to whether or not composite action should be counted upon in the negative moment areas of continuous spans. For this reason, in Article 3.9.4, this matter was left optional. It cannot be definitely settled until there has been opportunity to observe differences in the behavior of continuous composite structures with and without shear connectors over the negative moment areas.

The authors are deeply grateful to Mr. David for his thorough discussion of concretes with coefficients of thermal expansion and contraction differing from that of steel, including the subject of concretes with expansive characteristics. Mr. David refers in particular to composite construction. When a composite beam consisting of a steel girder and a concrete slab whose coefficient of thermal expansion is less than that of steel undergoes a change in temperature, the steel girder tends to change length by a greater amount than does the concrete slab. In order for the change in length of the slab to equal the change in length of the girder at their surface of contact, the slab must restrain the girder. This restraint is in the form of a shear force and a moment acting at the ends of the slab and the girder. The moment is formed by direct forces between the slab and the girder. The magnitude of the shear force and moment are determined from the requirements of compatibility of length changes and curvatures of the two components. As Mr. David points out, one compatibility requirement is that the curvature of the concrete equal the curvature of the steel. The other requirement is that the unit change in length of the concrete and the steel be equal at their surface of contact. It should be noted in the illustrative examples that the bending moment and the shear force are independent of the length of the member.

Mr. Perrin's discussion of the wind load provisions is pertinent and informative. It should be noted that the pressure of 35 lbs. per sq. ft. shown on the wind pressure map to which he refers was computed for a "fastest mile" of wind of about 80 miles per hour and for a shape factor of 1.3 intended to apply to buildings. The tests described in Mr. Perrin's reference (3) were made for the purpose of determining the transverse, longitudinal, and overturning wind forces to be used in the AASHO Specifications. By design, they included trusses of different solidity ratios representative of highway bridges. The increased drag in the case of multitruss bridges varies with the amount of shielding which is influenced by the truss spacing. It was not considered

necessary to attempt to cover these special cases in the specifications, especially since the economical use of multitrusses normally involves unusually wide structures in which the wind loading has relatively less influence on the design. Where Mr. Perrin refers to the drag coefficient for "round numbers" he probably means "round members."

At the present time there is very little information concerning the distribution of wheel loads through concrete slabs to prestressed concrete beams. Until more data are available, the distribution to steel stringers and beams is assumed to apply to prestressed beams also.

We are aware of the merits of the grid system floor, but the lengthy design computations which are necessary are a serious disadvantage to its adoption. We agree that deep diaphragms are effective in promoting load distribution and in preventing vibration.

Mr. Perrin's comments in regard to expansive characteristics of concrete are very much to the point and they corroborate Mr. David's statements.

ANALYSIS OF BRACED FRAMES[a]

Closure by Kurt H. Gerstle

KURT H. GERSTLE,[1] J. M. ASCE.—The author thanks Mr. Siegfried and Mr. Nelidov for their contributions.

Mr. Siegfried's formulation of distribution factors not only reduces the number of factors for an interior joint from nine to three, but also, by rewriting them in the following fashion, enables all unknowns to be expressed in terms of moments:

$$P_1 a_1 = \frac{K_1 a_1^2}{K_1 a_1^2 + K_2 a_2^2 + K_3 a_3^2} \Sigma P \cdot a$$

$$P_2 a_2 = \frac{K_2 a_2^2}{\Sigma K a^2} \Sigma P \cdot a \quad \left.\right\} \text{Eq. 4}$$

$$P_3 a_3 = \frac{K_3 a_3^2}{\Sigma K a^2} \Sigma P \cdot a$$

In these formulas all brace forces are expressed in terms of their effective moment $P_n \cdot a_n$ about Pin "O" (in Mr. Siegfried's figure). Two advantages result:

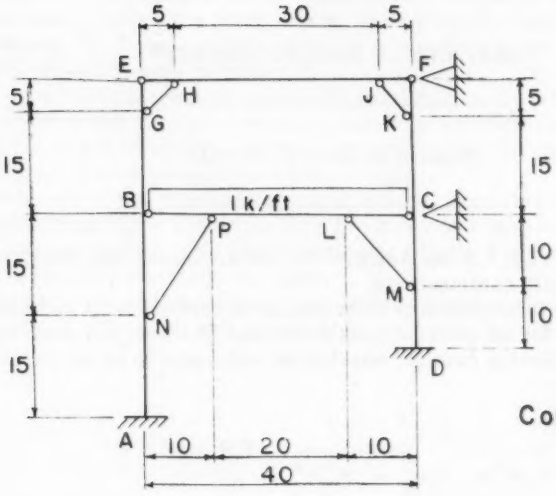
The sum of the distribution factors at a joint = 1.
The carry-over factors are dimensionless ratios.

Mr. Nelidov's solution, similar to the proposed general approach, illustrates one of the variations to which the method may be subjected, depending on the analyst's preference. Contrary to Mr. Nelidov's closing remark concerning the length of his solution, it is believed that with suitable tabulation and use of formulas such as those employed in the original calculation, the amount of time spent for the two versions might be comparable in the case of the single span frame. The original method might be preferred for the analysis of multi-span structures.

Another type of analysis for braced structures has been presented by Professor Seng-Lip See in "Civil Engineering", September 1958, p. 64.

The method has in the meantime been extended to the solution of multi-story braced frames with continuous columns, and the following example indicates the effects of continuity of the columns. The lower story is identical

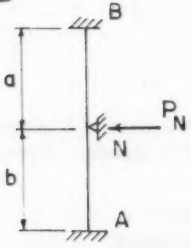
a. Proc. Paper 1560, March, 1958, by Kurt H. Gerstle.
1. Prof. of Civ. Eng., Univ. of Colorado, Boulder, Colo.



Constant I

Beam Constants:

AB:



Defl. Stiff. $K_N = \frac{P_N}{\Delta_N} = 7.11 \times \frac{EI}{10^3}$

Rot. Stiff. $K_B = \frac{M_B}{\theta_B} = 233 \times \frac{EI}{10^3}$

C.O.F.

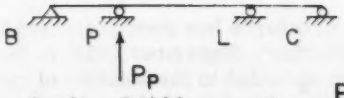
Due to P_N $C_{NB} = \frac{M_B}{P_N \cdot a} = +.25$

$C_{NA} = \frac{M_A}{P_N \cdot b} = -.25$

Due to M_B $C_{BN} = \frac{P_N \cdot a}{M_B} = +1.715$

$C_{BA} = \frac{M_A}{M_B} = -.143$

BC:

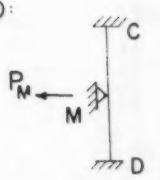


Defl. Stiff. $K_P = \frac{P_P}{\Delta_P} = 3.375 \times \frac{EI}{10^3}$

C.O.F. (See previous Example Prob.)

$C_{PL} = C_{LP} = +.778$

CD:



Defl. Stiff. (prop. to Mem. AB)

$$K_M = \frac{P_M}{\Delta_M} = \left(\frac{3}{2}\right)^3 \times 7.11 = 24.0 \times \frac{EI}{10^3}$$

Rot. Stiff. (prop. to Mem. AB)

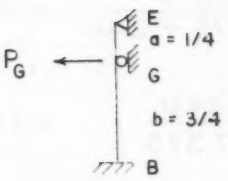
$$K = \frac{3}{2} \times 233 = 350 \times \frac{EI}{10^3}$$

C.O.F.'s: (Same as Mem. AB)

Due to P_M $C_{MC} = +.25$; $C_{MD} = -.25$

Due to M_C $C_{CM} = +1.715$; $C_{CD} = -.143$

BE and CF:



Defl. Stiff. $K_G = 17.5 \times \frac{EI}{10^3}$

Rot. Stiff. $K_B = 246 \times \frac{EI}{10^3}$

C.O.F. $C_{GB} = -.469$

$C_{BG} = -.833$

EF:

Defl. Stiff. $K_H = K_J = 6.53 \times \frac{EI}{10^3}$

C.O.F. $C_{HJ} = C_{JH} = .633$

Fixed Support Moments:

$F.E.M._{BP} = -17.81^K \times 10' = -178.1$ K-Ft

$F.E.M._{CL} = +17.81^K \times 10' = +178.1$ K-Ft

ST 3

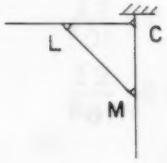
Distribution Factors (Siegfried's Method)

Joint B:

$$D.F._{BP} = \frac{3.375 \times 10^2}{3.375 \times 10^2 + 7.11 \times 15^2} = .174$$

$$D.F._{BN} = \frac{1600}{1937.5} = .826$$

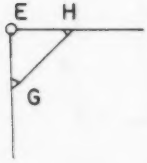
Joint C:



$$D.F._{CL} = \frac{3.375}{3.375 + 24.0} = .123$$

$$D.F._{CM} = \frac{24.0}{27.375} = .877$$

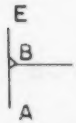
Joints E & F:



$$D.F._{EH} = D.F._{FJ} = \frac{6.53}{6.53 + 17.5} = .272$$

$$D.F._{EG} = D.F._{FK} = \frac{17.5}{24.03} = .728$$

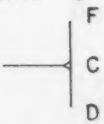
Joint B: (Column Rotation)



$$D.F._{BE} = \frac{246}{479} = .514$$

$$D.F._{BA} = \frac{233}{479} = .486$$

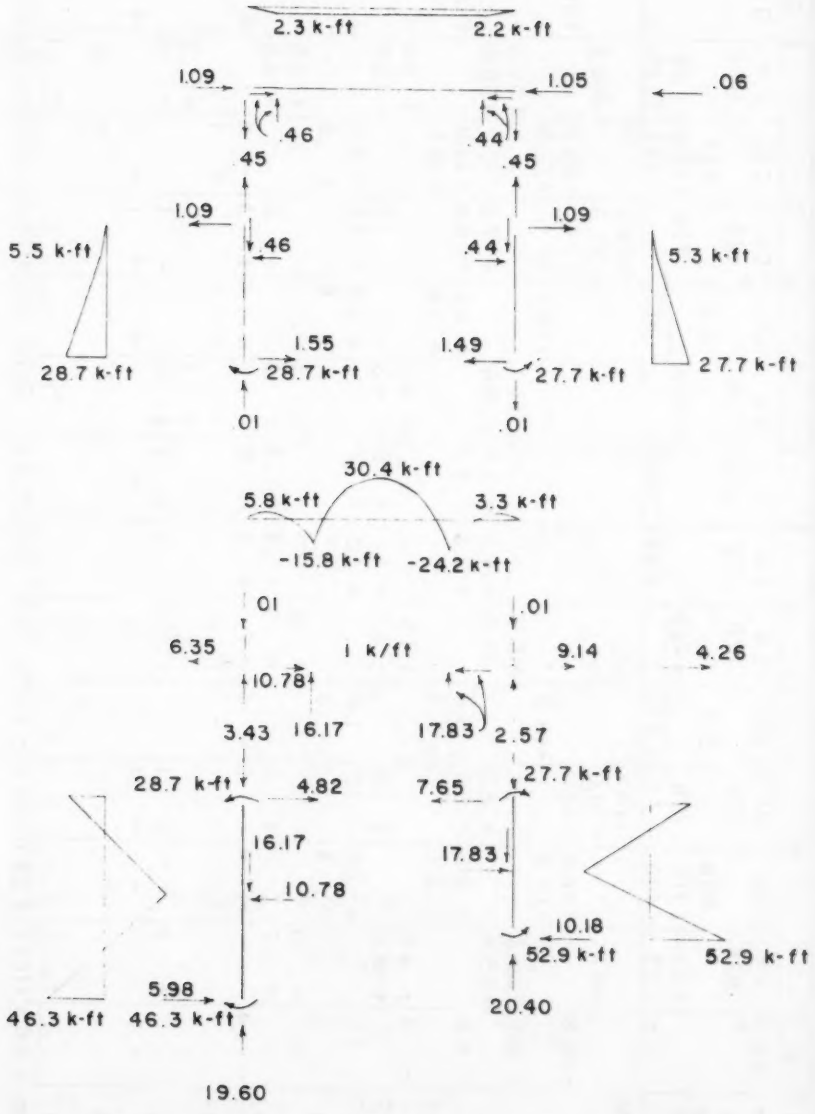
Joint C: (Column Rotation)



$$D.F._{CF} = \frac{246}{596} = .413$$

$$D.F._{CD} = \frac{350}{596} = .587$$

Joint	A	B				E		F		C				D
Mem.	AB	BN	BA	BP	BE	EG	EH	FJ	FK	CF	CL	CD	CM	DC
D.F. B		.826		.174		.728	.272	.272	.728		.123		.877	
D.F. M			.486		.514					.413		.587		
C.O.F.		+.25 ←	+1.715	.778	←	-.469			-.469	→	.778	+1.715	→ +.25	
"		← -.25	-.143		-.833	→	.633	← .633		→ -.833		-.143	-.25 →	
F.S.M.				-178.1							+178.1			
B_B		+147.1		+ 31.0							- 21.9		-156.2	
C	-36.8		+36.8	- 17.0							+ 24.1	-39.0		+39.0
B_M			-17.9		-18.9					+16.1		+22.9		
C	+ 2.4	-30.7				+15.7			- 13.4				+ 39.3	- 3.3
B_B		+39.4		+ 8.3		-11.4	- 4.3	+ 3.6	+ 9.8		- 7.8		- 55.6	
C	- 9.8		+ 9.8	- 6.1	- 5.4		+ 2.3	- 2.7		+ 4.6	+ 6.4	- 13.9		+ 13.9
B_M			- 2.1		- 2.3					+ 3.8		+ 5.5		
C	+ .3	- 3.6				+ 1.9			- 3.2				+ 9.4	- .8
B_B		+ 8.0		+ 1.7		- 3.1	- 1.1	+ 1.6	+ 4.3		- 1.9		- 13.9	
C	- 2.0		+ 2.0	- 1.5	- 1.4		+ 1.0	- .7		+ 2.0	+ 1.3	- 3.5		+ 3.5
B_M			- .3		- .3					+ .6		+ .9		
C	-	- .5				+ .2			- .5				+ 1.5	- .1
B_B		+ 1.7		+ .3		- .9	- .3	+ .3	+ .9		- .3		- 2.5	
C	- .4		+ .4	- .3	- .4		+ .2	- .2		+ .4	+ .3	- .6		+ .6
B_M			-		-					+ .1		+ .1		
C	-	-				-			- .1				+ .2	-
B_B		+ .2		+ .1		- .1	- .1	+ .1	+ .2		- .1		- .4	
C	-		-	- .1	-		-	-		+ .1	+ .1	- .1		+ .1
B_M			-		-					-		-		
C														
B_B		+ .1		-					-		-		- .1	
Mom.	- 46.3	+161.7	+ 28.7	-161.7	- 28.7	+ 2.3	- 2.3	+ 2.0	- 2.0	+ 27.7	+178.3	- 27.7	-178.3	+ 52.9



Note: All loads indicated are in kips

with the previously treated frame, so that the importance of column continuity may be evaluated by comparing results. The modification of Mr. Siegfried's method suggested by Eq. (4) will be followed.

The beam constants are evaluated considering the members fixed at the intermediate joints B and C. Carry over factors relate the effects of end moments and moments due to brace forces. Fixed Support Moments express the effect of the applied loads in terms of the Fixed Support Forces (explained in the original paper) times their lever arm about the joint. Two kinds of distributions are to be considered: First, the distribution of the unbalanced moments due to brace forces (denoted "B_B" in the tabular solution); the distribution factors for the operation are calculated according to Eq. (4). Secondly, the distribution of the unbalanced column end moments at Joints B and C in proportion to the rotational stiffnesses; this operation is denoted by "B_M" in the solution. The two types of distributions are performed as required to convergence.

The effects of side sway have not been considered in this solution. Two additional distributions for the two degrees of sway, and the solution of two shear conditions are required to complete the analysis. These operations are entirely analogous to the preceeding work.

The results of this problem may be compared to the results of the single story frame analysis (neglecting side sway). The similarity of forces and moments in both solutions suggest that it might be possible to neglect the effects of column continuity in preliminary analyses.

STRUCTURAL DYNAMICS IN EARTHQUAKE-RESISTANT DESIGN[a]

Discussions by Emilio Rosenblueth, M. J. Murphy, R. I. Skinner and K. M. Adams, and R. W. Binder

EMILIO ROSENBLUETH,[1] A. M. ASCE.—Mr. Blume's excellent contribution marks a milestone in engineering seismology. It gives valuable quantitative information about the influence of characteristics such as stiffness and damping on the seismic response of tall buildings. More important, it shows the feasibility of accurate dynamic analyses of these structures. It gives qualitative pictures of shear distributions in diverse types of buildings. And it points out most of the important variables in aseismic design.

The following comments should be construed as suggestions for future research rather than criticisms of the paper.

It is true that reliable designs can be obtained by subjecting an idealized structure to one or several records of actual earthquakes. The method is, however, more expensive and time-consuming than a dynamic analysis to compute the first few natural modes of vibration, followed by a combination of the stresses in these natural modes. And if the procedure for combining the natural modes is based on a probabilistic study that extrapolates to small probabilities of failure under the action of the universe of possible earthquakes, it will be more reliable than the analysis based on responses to a small number of earthquakes. Nevertheless the writer hesitates to recommend the available probabilistic methods, such as the assumption that, for small probabilities of failure, design stresses should equal the square root of the sum of squared responses in the natural modes:[1,2] there is yet insufficient substantiation of the theory; some indications[3] are that it may be satisfactory, whereas others[4] cast doubts on its adequacy.

Efforts intended to improve probabilistic theories of aseismic design seem therefore worthwhile. Particularly so because for more refined assumptions, say including ground coupling or considering explicitly more than one component of earthquakes, analyses will become much more complex, and some simplicity is saved by computing natural modes rather than seismic responses.

In the question of inelastic behavior, the writer agrees fully with Mr. Blume's advocation of elastic analysis to predict structural behavior under minor to moderate shocks, plus allowance for inelastic energy absorption under severe motions. Yet, there is at present no reliable method to make that allowance. The best that can be done is to design for stresses which are smaller than, and proportional to those which would develop under a major earthquake if the structure even then behaved elastically. There is

a. Proc. Paper 1695, July, 1958, by John A. Blume.

1. Research Prof., Inst. of Eng., Univ. of Mexico, Mexico City, Mex.

good reason in a multistory building for taking the design shear distribution proportional to one based on the assumption of elastic behavior, for if a few stories were underdesigned, or the rest overdesigned, those few stories would be overly taxed in energy absorption. But there is no reason to assume that underdesign of some frames, if the rest are sufficiently ductile, would not result in a better structural performance under a wide range of earthquake intensities than an elastic distribution, aside from resulting in greater economy.

Inelastic magnification of dynamic torsion is one phenomenon about which, as far as the writer is aware, nothing is known except that it may be important.

These considerations point out the immediate need for research of inelastic behavior of multistory buildings. But there is a stronger reason for this need. Differences in energy absorption capacity of various systems (unaided frames vs. X-braced frames vs. shear walls, etc.) are at present either ignored or taken in consideration through crude, almost intuitive variations in seismic coefficients. It is not going too far to say that a stage will likely be reached where explicit consideration of energy absorption capacity, coupled with a design based on drift limitations, will supplant stress computations. But a great deal more research is due before the time becomes ripe for such procedures.

The author recommends high working stresses associated with the admittance of high seismic forces. This tendency was followed in the 1957 revision of Mexico's Federal District Building Code[5] (although it is now generally felt that coefficients specified are too high for typical framed buildings). Doubtless the practice leads to a more uniform factor of safety and a better balanced design than the usual 1/3 increase in working stresses and the assumption of not more than moderate earthquake intensities, and should be advocated. But not without a word of caution concerning reductions in overturning moment. The fact that many structures have resisted apparently irresistible overturning moments is surely due in some measure to the reserve of their columns to resist high compressive forces, and this reserve comes largely from the use of low working stresses in static load and small increases for combined loading. In the use of high coefficients and stresses, reductions of overturning moments should be more conservative than in conventional practice. Indeed, allowable reductions in overturning moment were purposely left unspecified in the revision of the Federal District Code, for it was felt that the matter deserved considerable study, especially because working stresses were being increased at the same time that local soil conditions led to a proportionally greater response in the fundamental than in higher natural modes.

The problem of overturning moment reduction is more acute when results of dynamic analyses are preferred to static methods. In the latter the shear envelope, if correct for certain stories, is almost surely conservative for the rest, and the overturning moment computed is also conservative (aside from the fact that all maximum shears are unlikely to occur simultaneously). In a design based on dynamic analysis, all the shears may be essentially correct (again ignoring the unlikelihood of simultaneous occurrence), and the computed overturning moment not quite so conservative. It would be interesting to learn the author's recommendations on overturning moment reduction when analysis and design follow his other recommendations.

The author's assumed load-deformation curves for brick walls and tile partitions contradict some available data, which indicate a fairly flat portion after reaching the ultimate load.(6) Admittedly this part of load-deflection curves for walls must be very sensitive to the manner in which the wall is connected to the frame but this fact merely points out the need for considerable research. The energy absorbing capacity of walls and partitions with a flat portion in the load-deflection diagram is especially interesting in the plateau scheme proposed by the author.

In brief, without detracting from the paper's exceptional value, the writer believes probabilistic methods more promising than analyses based on responses to a short number of earthquakes. He also believes that a practice more conservative than is usually accepted must be followed with respect to overturning moments when using the author's proposed criteria of design.

Agreeing with the author's call for consideration of energy absorption beyond the elastic limit, the writer feels there is no rational way to make that allowance. Research is urgently needed in this connection.

The author's load-deflection curves for walls and partitions may be correct under some conditions. But sometimes they may grossly underestimate the energy absorbing capacity of these elements. This is one of the many pressing problems of aseismic design meriting intensive research. It would be valuable for the profession to learn the author's views on these questions as well as on other important matters which he either omits in his paper or barely touches upon, for example, accidental torsion, ground coupling, and the combined effects of the six earthquake components (three displacements and three rotations).

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3. Tung, T. P. and Newmark, N. M., "Shears in a Tall Building Subjected to Strong Motion Earthquakes," ibid, Chapter 10.
4. Bustamente, J. and Veletsos, A., unpublished study of miltistory buildings.
5. Chapt. 41.1 of Reglamento de las Construcciones y de los Servicios Urbanos en el Distrito Federal, Gaceta Oficial del Departamento del D. F., Revised Sept. 1957, Federal District Dept., México, D. F.
6. Benjamin, J. R. and Williams, H. A., "The Behavior of One-Story Brick Shear Walls", Proc. ASCE, Vol. 84, No. ST4 (July 1958) Part 1, paper 1723 (30 pp).

M. J. MURPHY,[1] R. I. SKINNER,[1] and K. M. ADAMS.[1]—The analysis of the 15-storey Alexander building does much to explain the difference in actual performance of a real traditional structure, and the performance that elastic dynamic theory predicts for a structure whose cladding strength and energy-absorption characteristics have been ignored. This difference has been a stumbling block for some time and has led many engineers to take a cautious view of dynamic theory in general. If one accepts the author's explanation for the real source of seismic strength in a traditional building (and the evidence seems to us to be incontrovertible) it becomes necessary to take a closer look at the modern building in which one dispenses with crushable masonry walls and is at pains to prevent architectural damage.

If the structural engineer can state the floor masses and interstorey stiffnesses correctly, dynamic analysis can provide the interstorey shears dependent on only one variable—the damping. It has been customary to assume 10 - 20% viscous damping as a realistic figure. But where in nature does viscous damping come from? Air damping would seem to be the only thing which is truly viscous but unfortunately negligible. The internal viscous damping of materials which has been postulated is very small indeed. Let us try to list all possible sources of energy loss which may fairly be described as not resulting in architectural damage.

Foundation Damping

Actual loss of energy through movement of the soil is possible but it seems unlikely to be large. Where a building is supported on long piles, however, the upper ends of these piles must move through the soil. This could well provide significant damping but as yet just how much remains to be assessed. Foundations on rock, of course, would not have this form of damping.

Foundation Yield

The static bearing qualities of soils have been thoroughly examined but as yet no dynamic data are available. Even if soils showed a readiness to yield under dynamic loads, few engineers would care to appeal to this source of energy absorption, postulating as it does the possibility of unequal settlement.

Loss of Energy by Radiation into the Ground

Normally there is a large discontinuity in the impedance to shear-wave propagation at the base of a building. Hence, whenever the base of the building is small compared to the wavelength in the surrounding material, little radiation of energy will occur.

Hysteretic Damping of Materials

Below yield point, steel has practically none. Concrete has some, but nothing as large as 10% damping figure.

Frame Yield

This represents a major possibility for energy absorption whether the frame is of steel or reinforced concrete and this Section has recently

1. Dept. of Scientific and Industrial Research, Dominion Physical Lab., Lower Hutt, New Zealand.

submitted to the publishers an initial enquiry into the amounts of yield to be expected with different building periods and different levels of yield force. If, as Mr. Blume and the authors think, yield represents a major source of seismic strength, it is as well to note at this stage that prestressed concrete, at the present stage of development, has virtually no yield at all, and its indiscriminate use in seismic areas is, to say the least, suspect. The subject of yield has only been touched on so far and there are certainly not sufficient data available to provide a basis for a design procedure.

Friction Damping

The possibility of providing quite large friction damping in a modern building has been brought to our attention by Dr. B. H. Falconer, Consulting Engineer, Wellington, New Zealand. Interior walls and weather cladding, which modern practice requires should be free of the frame, could well be attached by means of friction clamps arranged to slip at a value below that which would cause architectural damage. It would also be possible to provide some really healthy damping at the bottom of the building with a specially constructed friction device.

Reviewing the position then, the traditional building owes its good seismic reputation to the strength and energy absorption characteristics at failure of the masonry panels and architectural features. The modern structure is either eliminating these features or attaching them to the frame in such a manner that architectural damage is not sustained. In short, the design has become so clean that the basic assumptions of dynamic analysis are justified and the answers (showing very high shears even at 20% damping) are valid. It becomes a matter of some urgency then to examine:

(a) The amount of damping that can be expected from piled foundations.
(b) The amount of damping that can be built in with friction panels.
(c) The amount of yield that can be allowed between stories and still leave Euler stability in the columns and services unimpaired.
(d) To make a dynamic analysis of the design features from (a), (b) and (c) and see that the designed building, within the limits of its known properties, is capable of surviving an earthquake.

Mr. Blume makes short reference to bearing-wall structures which deflect not in shear but in bending. This is a point that worries us as we are not yet able to analyse them and there are many quite tall ones in New Zealand (up to 10 stories). They are inherently stronger than frame structures but unfortunately it is not possible to postulate yield damping.

R. W. BINDER,[1] M. ASCE.—A review of earthquake resistant design procedures will show that some 30 years ago, although the earthquake problem was understood to be one of dynamics, the approach was nevertheless primarily one of statics with special details incorporated to obtain a reasonable earthquake resistant structure. In other words, it might be said that earthquake resistant design at that time was in most instances more of an art than it was a science.

1. Chf. Engr., Fabricated Steel Construction, Bethlehem Pacific Coast Steel Corp., Torrance, Calif.

In the early 1940's considerable research studies brought about a change in thinking, and a pseudo dynamic design approach came into being. An excellent paper on the earthquake problem prior to 1952 is that of Lateral Forces of Earthquakes and Wind.(1) The practical application of design from about 1940 until 1952 stimulated further important research on the dynamic aspects of the earthquake problem. Thus, today those who are familiar with the research work may consider earthquake resistant design as much a science as an art.

The paper by Mr. Blume not only contributes to the theory, but also in a most enlightening manner develops important factors of the art of earthquake resistant design. The manner in which this subject matter is developed makes it a most important paper and one which should be studied very carefully by those who desire to have a better insight into the subject of earthquake resistant design.

The area in which an earthquake occurs will still continue to be a huge laboratory for the investigation of the effects of earthquake shock. A study of Mr. Blume's paper is such that an observer can readily avoid the pitfall of drawing intuitive conclusions and being influenced by repeated statements which have become cloaked with authority.

About the time that Mr. Blume was presenting his paper, the writer was also presenting a paper(2) on the Mexico earthquake of July, 1957. Mr. Blume states, "For a structure or a member to withstand violent abuse such as the impact and motion of a major earthquake it must, in the language of athletics or the military, 'be able to take it'; it must be tough, and it must be well tied together, to act as and to remain a unit. Toughness implies the ability to absorb punishment and still retain integrity and resistance against additional punishment." The writer in his paper wrote, "This type of construction is similar in one respect to the reactions of football players. The All-American player, when tackled hard, is almost always relaxed, and thus will suffer no injury; however, the player who stiffens up stands a good chance of being cracked up." Putting the two statements together, it would seem that a good structure, like a good athlete, should, as necessary, be capable of rolling with the punch and be tough enough to absorb a maximum of punishment.

Figs. 6, 7 and 8 and companion tables represent an important compilation and cover some aspects of the science and the art of earthquake resistant design, pointing up the fact that the designer should have in mind not one design but rather two designs in order to achieve an economical and sound structure. The energy absorption of stiff or rigid elements at low strains may involve other problems in considering the overall design. Variable stiffness resulting from construction differences, temperature changes, etc. may result in torsional effects. The complete lateral force resisting structural frame in this case is of extreme value, as it represents many stress paths to take up torsional effects and the vibrations resulting from a strong shock.

The formula for K_v (analog) on page 22 is obviously a typographical error, and one of the x's in all probability should be a plus sign.

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WIND FORCES ON STRUCTURES: INTRODUCTION AND HISTORY[a]

Discussion by C. Scruton

C. SCRUTON.[1]—Civil engineers will welcome the efforts of the ASCE Committee on Wind Forces to correlate and summarise existing data relating to wind effects on structures. Most of the information comes from wind-tunnel studies reported from many sources, often from ad hoc rather than systematic investigations, and the task of collating this information and preparing a simple code of practice to adequately cover the diversity of structural types and designs is most difficult.

a. Proc. Paper 1707, July, 1958, by J. M. Biggs.

1. Aerodynamics Div., National Physical Lab., Middlesex, England.

WIND FORCES ON STRUCTURES: NATURE OF THE WIND[a]

Discussions by Herbert S. Saffir, W. R. Schriever, Donald C. Bunting, R. C. Gentry, A. G. Davenport, Henri Perrin and C. Scruton

HERBERT S. SAFFIR,[1] M. ASCE.—There appears to be no basis to support Mr. Sherlock's comments that a decreasing pressure and velocity gradient with increasing height should be used for tropical hurricanes of over 130 M.P.H. velocity.

The consensus seems to be that wind velocity in hurricanes (as well as in non-tropical storms) increases from the surface up to about 3,000 feet; for the next several thousand feet hurricane winds show very little change; throughout the upper layers of the vortex there is a rapid decrease with elevation, and with heights of 40,000 feet or so, the cyclone often can not be detected. Possibly the wind data to which the author referred pertained to some high elevation and not to the lower levels in which the structural engineers are most interested.

W. R. SCHRIEVER.[2]—According to Circular 2606 of the Meteorological Division the wind speeds on the night of October 15 and 16, 1954 were high but not record breaking. In general the highest speeds were observed to the east of the path of the storm centre; that is, from Toronto along the north shore of Lake Ontario and east of Ottawa. The greatest wind speeds were reported near Toronto where the centre of the storm passed just to the east of Malton Airport. Malton Airport reported a maximum hourly mileage of 54 with a maximum one minute wind of 58 mph (direction West SouthWest) and a maximum gust of 78 mph. These are the highest speeds reported for Hurricane Hazel in Ontario. As far as I know no information on the increase of velocity with height is available.

DONALD C. BUNTING.[3]—With reference to gradient winds as compared to observed surface winds in hurricanes there is most likely a similar relationship as in extra-tropical cyclones, that is, an increase in speed with height up to the gradient level. One explanation for the discrepancy shown in the tables in the writer's paper in Weatherwise is that the computed maximum gradient winds were obtained from averaging data over 10- to 15-minute intervals so that it would be improper to compare observed 1-minute maximum winds with computed 15-minute maximum gradient winds. Naturally, the longer-time-

a. Proc. Paper 1708, July, 1958, by Robert H. Sherlock.
1. Asst. County Engr., Dade County, Miami, Fla.
2. National Research Council, Building Structures Section, Ottawa, Canada.
3. Asst. Prof., Dept. of Physical Sciences, Univ. of Florida, Gainesville, Fla.

interval average winds will always show slower speeds than the shorter-time average winds. This can be seen from the data in my paper where in two hurricanes the 1-minute maximum surface winds are given as 155 and 153 mph whereas the same storms show the 5-minute maximum surface wind to be 121 and 132 mph respectively. If this were further extrapolated to a 10- or 15-minute interval it can be seen that the maximum surface winds would be reduced to something less than the maximum computed gradient winds of 102 and 99 mph for the same two storms. On the average though, there would not be a great deal of difference at coastal locations between the maximum average surface winds and the maximum average gradient winds if both were taken over the same time interval. This is due to the fact that there is only a small amount of frictional effect present over the open sea and other factors which would cause a difference are considered to be negligible in the high speed zone of a hurricane.

The 1-minute average surface winds given in the table in the writer's paper would be considerably influenced by peak-gusts during the short time interval. There has been evidence that these gusts at surface levels are due mainly to bubbles of air brought down by turbulence from rapidly moving layers at some higher levels. Although these bubbles most likely originate in the frictional level (i.e. below the gradient level), it is generally believed that their origin is high enough and the loss of momentum small enough that the gust speeds will essentially be about the same as the gradient wind speeds. Hence, the peak gusts measured at the surface levels may, without serious error, be considered the same as the gradient winds.

There is another strong wind which Mr. Sherlock does not mention that the writer would like to see included. This is the so-called "first gust" associated with most all thunderstorms. The writer has seen these winds reach speeds up to 70 mph and they may certainly cause considerable damage to flimsy structures. They are, of course, of only short duration but their suddenness of appearance may sometimes make them dangerous.

In hurricanes there is another type of wind called a "plow wind" similar to the first gusts in thunderstorms that has caused a great deal of damage during hurricanes. In Florida there are also tornadoes often associated with hurricanes that have caused great destruction to buildings in some of the Great Hurricanes which have passed over the state.

Of special interest is the Hydrometeorological Report No. 32 entitled Characteristics of United States Hurricanes Pertinent to Levee Design for Lake Okeechobee, Florida issued in March 1954 by the Weather Bureau. It was from this report that the calculated tabular data was obtained for the writer's paper. Chapters III and IV are especially pertinent to this work.

R. CECIL GENTRY.[1]—The following discussion represents the personal opinion of the writer and not the official viewpoint of the Weather Bureau.

Professor Sherlock has prepared an excellent well written paper which contains a lot of good information on winds. However in the section dealing with the variation of wind velocity with height below 1500 feet in hurricanes faulty information from the extremely sparse available data may have led him into drawing conclusions which are not justified.

1. Acting Director, National Hurricane Research Project, Weather Bureau, U. S. Dept. of Commerce, West Palm Beach, Fla.

Contrary to Professor Sherlock's conclusion it would seem that the wind speed must increase in height from the surface of the earth up to a level where the influence of surface frictional drags become relatively negligible. Unfortunately there are very little actual data that can be used to determine the variation of wind with height for the lower layers in the area of strongest winds. There is supporting evidence, however. The winds measured by instruments on the tower at Brookhaven during the passage of hurricanes Edna and Carol showed winds increasing with height. Winds measured at rawin stations near the area of hurricane force winds usually show speeds increasing with height for the first 500 to 1000 feet. In most cases the strongest winds measured when a hurricane crossed the coastline were measured at a station where the elevation of the anemometer was well above the 30 ft. level, e.g. in the three cases quoted by Professor Sherlock from Mr. Bunting's paper, the strongest winds were measured at lighthouses with the anemometers at levels higher than 100 ft. rather than at one of the stations where the anemometer was at 20 to 40 ft. elevation. This happens so often that it can hardly be considered merely a coincidence. To further illustrate the later point the following tabulation was prepared which shows the maximum wind recorded at two Miami, Florida stations during the passage of hurricanes near that point. The numbers in parentheses give the elevations of the wind measuring equipment. Note in each case that the station with the higher elevation recorded the higher velocity. These data unfortunately cannot be used satisfactorily for determining the variation of the wind with height because of the different exposure of the two stations and the distance between them (approximately 5 miles).

Fastest Mile Recorded by Miami Weather Bureau		Dates of Hurricanes
Airport Station	City Office	
94 (50)	107 (249)	9/15/45
75 (50)	76 (229)	9/22/48
68 (50)	86 (229)	10/ 5/48
46 (50)	60 (229)	9/26/49
80 (50)	122 (229)	10/17-18/50

Thus the indirect evidence available plus consideration of the effect that frictional drags must have on air velocities both point to the conclusion that at least in the first few hundred feet above the ground, the wind speed must increase with height and the percentage increase may be appreciable.

The data that the author quoted from meteorological publications and from which he concluded that the wind velocities are either constant or increase with height are either fallacious or very unreliable. The maximum one minute velocities cited from the paper by Mr. Bunting were measured at Dry Tortugas, Hillsboro Light and Jupiter Light. The elevation of the anemometers at these three stations were respectively 166 ft, 146 ft and 106 ft rather than the 30 to 40 ft which the author assumed for lack of better information.

The "Extreme gradient wind" used for these same three storms were winds Mr. Bunting had quoted from another source and are at best based on very little data. Estimating gradient winds accurately is very difficult under the best of circumstances, and is almost impossible to do with the data available for the three hurricanes Mr. Bunting was discussing. The big problem is lack of data. The maximum winds in hurricanes frequently occur in bands less than 10 miles wide. Thus to estimate the maximum gradient wind, one would

need to know the pressure gradient through this narrow band of maximum winds. Since the pressure data were available only from stations spaced 50 to 100 miles apart, an estimate of the maximum pressure gradient (over a 10 mile distance) is simply a guess. In the work quoted by Mr. Bunting and referred to by Professor Sherlock, the so-called gradient winds were really estimated cyclostrophic winds (i.e. pressure gradient balanced by centrifugal force due to the curvature of the wind path). Furthermore the wind estimates were based on a formula that was derived by empirical procedures in which an attempt was made to relate the calculated wind to the strongest observed winds averaged over a 10 minute period—not a one minute period. In deriving this formula it was assumed that the pressure gradient was symmetrical around the hurricane. Actually nearly all evidence available shows that in a moving hurricane the maximum winds are not symmetrically distributed about the storm, but are concentrated on one side of the center.

Two other points in the paper deserve comment. Besides the hurricanes forming in the Western Caribbean and near the Cape Verde Islands, continental areas of the United States are affected even more frequently by tropical cyclones originating in the Gulf of Mexico, near and east of the Bahama Islands, and in the area just to the east of the Leeward and Windward Islands.

With regard to the altitude of the most severe turbulence very few data are available that furnish answers. Experience of the aircraft reconnaissance crews in recent years indicate that the turbulence is less in the 300-500 ft layer than in the 1000-2000 ft layer. Nearly all of this reconnaissance experience was gathered from flights over the water. Since the frictional affects may vary so greatly from the water to the land surface it is not at all certain that the altitude of maximum turbulence would be over land.

A. G. DAVENPORT.[1]—Professor Sherlock's most recent, excellent and informative paper on the nature of the wind has been read by the writer with the greatest interest particularly in view of Professor Sherlock's many notable contributions to this subject. Naturally enough, a paper which embraces in such short span so broad a subject of necessity leaves some avenues unexplored. One of these avenues concerns the effects of stability and surface roughness on the variation of wind velocity with height which Professor Sherlock does not discuss beyond saying that "the variation of wind velocity with height will be influenced by the type of storm and the terrain or water area over which the wind has passed". Closer consideration of the full implications of these factors, stability and surface roughness, may lead to recommendations which are not always identical with Professor Sherlock's.

In a paper submitted for publication,(1) the writer has attempted to evaluate quantitatively these effects of surface roughness and stability insofar as they influence the determination of design wind velocities. This study (based largely on information already published) has suggested the following general inferences and conclusions. First, with respect to the influence of stability (determined by the lapse rate or variation of temperature with height), it would appear that

(1) Severe local storms, such as thunderstorms, frontal squalls (and perhaps other storms such as hurricanes in their early incipient

1. National Research Council, Div. of Building Research, Ottawa, Canada.

stages) are extremely unstable causing continuous interchange of the faster moving upper air and retarded air nearer the ground. Consequently the increase of mean velocity with height is very small. The frictional characteristics of the ground surface may have almost negligible effects on the velocity profile.

(2) Large scale mature storms of either tropical or extra-tropical description exhibit nearly neutral stability with no marked tendency for violent thermal interchange. The dominating influence on the velocity profile in these storms is not the stability but the surface roughness.

The question therefore arises as to how great an effect does surface roughness have on the wind velocity profile in storms of the latter category, which are probably the more important. In comparison with the vast amount which has been written in engineering papers on this general subject, little attention appears to have been given to this particular question and yet it appears to be paramount in the accurate evaluation of wind velocities.

It should first be emphasized that what is referred to by the term "surface roughness" is neither the shielding due to individual obstacles nor the orographic effects influencing the airflow in mountain regions but the cumulative statistical drag effect of obstructions on the wind. Thus the surface roughness is related to the density, size and height of the buildings, trees, vegetation, rocks, etc., on the ground around and over which the wind must flow: it will be a minimum over the ocean and a maximum in a large city.

Fig. 2A of Professor Sherlock's paper (published in error as Fig. 3) illustrates wind profiles obtained from regions exhibiting three distinctly different types of ground roughness and yielding three distinctly different rates of increase of wind velocity with height. The two profiles, O and P, shown in Fig. 2A, are taken from hurricane records at Brookhaven. In this vicinity the countryside is "fairly level and is uniformly covered with scrub oak and pine to a height of from 25 ft. to 30 ft. above ground".(2) In these profiles the increase in velocity with height correspond to power laws with exponents of between 1/4 and 1/3 and Singer and Smith(2) state "The fact that a study of thirteen recent storms indicated a mean value of the exponent of .250 shows that these values are typical of the profiles during strong winds in this location". The increase of velocity with height is therefore consistently greater than that found in Professor Sherlock's own outstanding investigations of storm winds at Ann Arbor, Michigan (curves E & F) over "fairly level, open farm land". In these the 1/7th law fitted the records well. This profile in turn represents a greater increase in velocity with height than that of curve Q suggested by hurricane records from three Florida lighthouses fully exposed to the open water, one of which, Dry Tortugas, is situated on a small island more than one hundred miles from the Florida mainland.

Curves A, B, and C of the same diagram refer to velocity profiles obtained at the Scilly Islands some thirty miles to the southwest of Lands End in Great Britain. Since the wind velocities amount to no more than gentle breezes and the prevailing stability conditions are not known, these may be "heterogeneous observations" and therefore not comparable. The steepness of the profiles suggests however, that they may not be unrepresentative of gale winds blowing over the surface of open water.

Other published records (listed in Ref. 1) suggest even more positively the general influence of surface roughness on the wind velocity profile in high winds and in storms of close to neutral stability. If the increase of mean

velocity ("fastest mile") with height z is represented by the power law then, within the range of heights 30 to 1000 ft. (of interest to the structural engineer) the values of the exponent corresponding to different conditions of surface roughness would appear, generally speaking, to be as follows:

Description of Terrain	Power Law Exponent	Mean Velocity at 100' As Percentage of Gradient Velocity
Open country, flat coastal belts, small islands in large bodies of water, prairie grassland, tundra	1/7	70%
Wooded countryside parkland, towns, outskirts of large cities, rough coastal belts	1/3.5	50%
Centre of large cities	1/2.5	33%

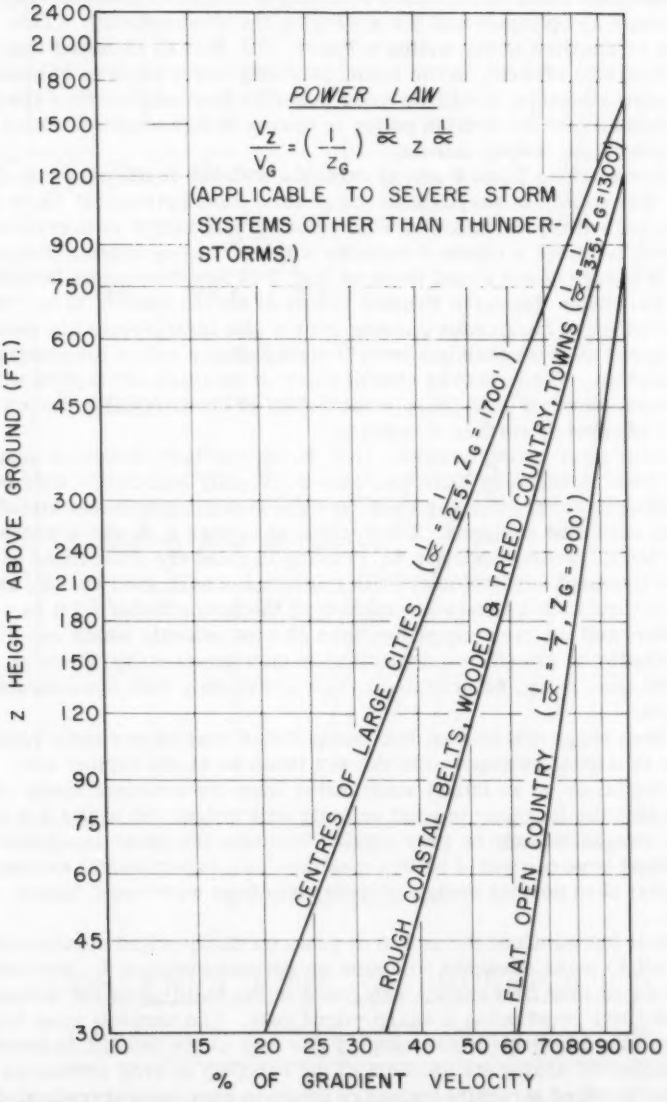
The last column of this table gives estimates of the retardation produced by the different types of terrain, as shown by the ratios of the mean velocity at 100-ft. height to the gradient velocity which is unaffected by the surface friction. All these figures refer specifically to mean velocity profiles over level (although not necessarily "open") terrain. These power laws are illustrated in Fig. 1.

This exploratory discussion bears directly on some of Professor Sherlock's recommendations. In regard to the first recommendation, it is certainly agreed that the standard of reference should be level country, but the further condition that it should be strictly open has debatable advantages. If it is possible to take account of the influence of surface roughness (as this discussion suggests it might be), many valuable records might be included from areas which, although level, may be other than open.

The effects of surface roughness on the wind velocity can be at least as great as the "influence of shielding or of deflection and channelling due to unusual topography". It is therefore logical to suggest that cognizance should be taken of the effect of surface roughness.

It could be pointed out that in at least two countries—Japan and Russia—consideration is given to the effects of surface roughness on the wind velocity profile. In Japan, which is subject to typhoon wind of similar character to hurricanes, the power law exponent of 1/3 is suggested for towns and cities and 1/5 at the coast.(3) In Russia, a velocity profile with an equivalent power law exponent of roughly 1/4 is specified for the whole territory of the USSR with the exception of the coastal belts of inland seas and the Black Sea, where the equivalents of 1/5 and 1/8.5 power law exponents are specified respectively.(4)

With regard to the effects of unusual topography, it would be helpful to designers to have available, information that would assist in estimating whether the amplification which may arise due to this is 50, 100 or 200 per cent. The use of anemometers during initial site exploration should also be considered in some cases as a means of determining amplification factors when the topography suggests that these may be large.



INCREASE OF VELOCITY WITH HEIGHT OVER LEVEL GROUND FOR THREE DIFFERENT TYPES OF SURFACE ROUGHNESS ACCORDING TO THE POWER LAW.

FIGURE 1

In regard to the third recommendation, the writer endorses the use of the fastest observed mile. This, apart from any availability consideration represents perhaps an optimum unit for averaging the wind velocity. (This statement is justified in the writer's Paper '1'.) It is to be hoped that in Canada, where at present, in the main, only long-term records of hourly mileages are available, it will be found possible to emulate either the American or alternatively the British policy in basing design wind velocities on the fastest mile or the fastest minute.

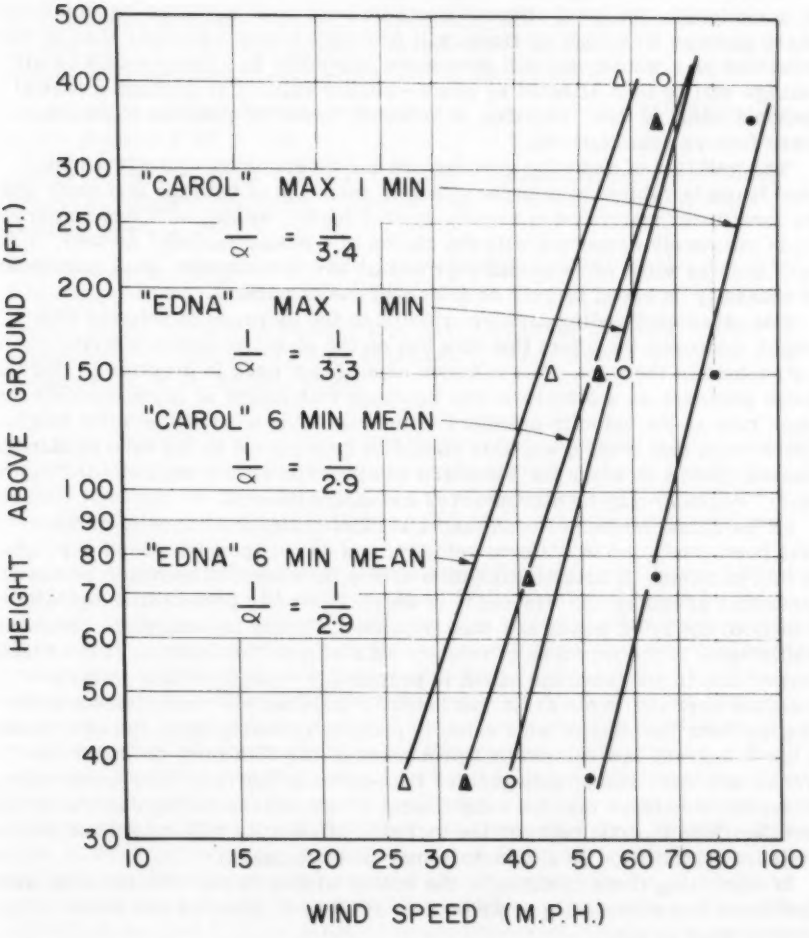
Recommendations 5 and 6 are at variance with the findings of this discussion. The writer is skeptical of the general assumption that "it is reasonable to disregard the variation of wind velocity with height in hurricane regions and to accept a constant velocity with increasing height. Professor Sherlock's curves O and P and those of Fig. 2 of this discussion (stated by the National Hurricane Research Project report of March 1957[5] to be "the only known instances of hurricanes passing over a site instrumented to measure the wind speed at more than one level") unmistakably refute the generality of this assumption. These curves exhibit rates of increase of velocity with height almost identical with those exhibited by extra-tropical cyclones under similar conditions of surface roughness.

The writer does agree, however, that during the early incipient unstable stages of a hurricane very little increase of velocity with height may prevail: it is doubtful, however, whether these are the general conditions for which structures should be designed. Evidently it is curves J, K and L which have generally led to recommendation 6. Bearing in mind the difficulties of estimating the gradient wind velocity (which increases with wind speed), and the unknown factors such as the over-running of the anemometer (if it is a cup anemometer) and the large amplifications of wind velocity which can arise if the anemometer is situated on a building or on a promontory, these records can, for the time being, be accepted by the writer only with considerable reservation.

It has been suggested in this discussion that it may be perfectly justifiable to assume in a location exposed to the sea (such as at the Jupiter and Hillsboro Light) or on an island many miles from the mainland (such as Dry Tortugas) that the increase in wind velocity with height due to the low value of surface roughness may be very small. To make the same assumption further inland however would be incompatible with experimental evidence and, unless higher than normal design surface velocities were used, highly dangerous.

So little is known about the action of gusts on sharp edged structures that it is difficult to make adequate comment on recommendation 7. Professor Sherlock has several times made reference to the build-up of lift forces on aerofoil sections penetrating a sharp-edged gust. The aerofoil must travel roughly six chord lengths before even 95 per cent of the full lift is developed. The not altogether analogous problem of the build-up of drag pressures on bluff or sharp-edged structures appears never to have been investigated, except analytically in the case of a cylinder at very low Reynolds numbers.[9]* It may be anticipated that the size will be one of several governing factors in a manner indicated by Professor Sherlock's suggested gust factors. Recognition of this, the writer considers, is a big step forward.

* The writer is at present attempting such a study at the University of Bristol.



OBSERVATIONS OF HURRICANES "CAROL" & "EDNA" (1954) AT BROOKHAVEN LONG ISLAND. (TERRAIN: FLAT COUNTRY WITH SCRUB TREES)

FIGURE 2

There are other important factors apart from size, however, which should be considered: the local effect of gusts on individual surfaces (especially at sharp corners with high suctions) will probably be much greater than on the structure as a whole; internal pressures depending to a large extent on air leakage will be less affected by gusts—window rattling is perhaps a partial demonstration of this. One day, it is hoped, it will be possible to evaluate these factors quantitatively.

The problem of deducing gust factors for determining equivalent static wind loads is related also to the dynamic response of both the structure and its constituent materials to rapidly applied loads: as Horne[6] has remarked "it is intimately connected with the choice of a design method" as well. Until such time as some of these many problems are investigated, gust factors will of necessity be based largely on informed guess work.

One other outstanding problem related to the increase of velocity with height, concerns the effect that this has on the static pressure distribution on a structure in the wind. The common assumption used in practice is that the static pressure on a structure will increase with height at approximately the same rate as the velocity pressure of the oncoming wind at the same height. There is no real reason why this should be true except in the case of slender trussed towers in which the windward and leeward vortex regions (or "dead water" regions) may be disconnected and discontinuous.

Of the large number of wind tunnel studies made, the majority of these have been conducted in uniform velocity, and do not therefore shed any light on this problem. A notable exception to this, however, is provided by the experiments of Bailey and Vincent[7] in which a velocity profile corresponding closely to the 1/7th power law was simulated. These investigators comment that in spite of the increase of velocity with height "the analysis of the results showed that in all cases the effect is practically equivalent to a uniform pressure over the whole projected height". Rathbun's[8] experiments at the Empire State Building in wind velocity profiles probably corresponding closer to the 1/2 power law showed that pressures at the 36th floor did not differ greatly and were sometimes greater than those at the 75th. For these reasons the writer considers that the relationship which is assumed by current design specifications to exist between the increase of velocity with height and static pressure distribution on a structure may be inadequate.

In submitting these comments, the writer wishes to pay tribute to the many significant and stimulating contributions Professor Sherlock has made to the study of wind action.

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HENRI PERRIN,[1] A. M. ASCE.—The decrease in wind velocity with height has already been observed under some circumstances,(1) but it has been generally assumed by most foreign and American codes that the wind force increases up to an elevation of 1,000 to 1,500 feet, above which a constant value shall be used.

The author recommends to use a constant basic wind velocity when dealing with a tropical cyclone. The writer would like to add that the gust factor should also be assumed constant, since the gust factor varies inversely with altitude, but in a more general way, inversely with wind velocity.(2) The wind pressure thus obtained will be constant up to the elevation where a wind pressure, based on the seventh power law, with due correction for varying gust factor, and starting from a basic wind velocity equal to 75 MPH, will govern. A suggested minimum value for the constant gust factor is 1.10.

The recommended gust factors are 1.10 and 1.30, and depend on the size of the structure, implying the use of the former value for a guyed tower. The suggested wind loading for a guyed tower, as shown on Fig. 4, includes a reduction of 25% corresponding roughly to a gust factor of 1.10 being omitted. However it has been proposed(3) that the cantilevering top of a guyed tower and its connections should be designed for a higher wind loading. The writer is of the opinion that for this weak, although important, part of the structure, a gust factor up to 1.30 may be used. The final choice of a gust factor for a guyed tower should however be also conditioned by the size and response time of the whole structure.

An interesting recommendation is outlined in the French(4) and Belgian(5) Codes concerning the stability of a structure. The wind loadings are multiplied by 5/3 and 2, respectively, in order to check the stability and buckling factors. Such a consideration is important for structures subjected mainly to wind loading which is not always in a direct ratio to the stresses, as for a guyed tower.

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1. Sr. Designer, Barstow, Mulligan & Vollmer, New York, N. Y.

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C. SCRUTON.[1]—Perhaps the greatest difficulty which confronts the designer is the uncertainty in the prediction of the most severe conditions to be encountered at the site, and in the assessment of the minimum gust period of significance in structural design. It is this latter aspect which the writer would like to see developed in more detail.

1. Aerodynamics Div., National Physical Lab., Middlesex, England.

WIND FORCES ON STRUCTURES: FUNDAMENTAL CONSIDERATIONS[a]

Discussions by C. Scruton and Henri Perrin

C. SCRUTON.[1]—Messrs. Woodruff and Kozak use the terms "aerostatic" and "aerodynamic" to denote, respectively, the steady and unsteady forces due to wind. These uses conflict with the usage of similar terms by the aeronautical engineer and with the use of the analogous terms "hydrostatic" and "hydrodynamic" by the civil engineer. To avoid confusion due to the use of different terminologies the writer would like to see other terms found and suggests that "steady aerodynamic force" and "unsteady aerodynamic force" might be acceptable as less ambiguous phraseology. Although one cause of the oscillations in wind of flexible bluff bodies such as tall stacks is attributable in a general way to vortex-excitation, little is as yet known of the more detailed mechanism of the exciting force, and there is insufficient data from which to obtain reliable predictions of its magnitude, especially as supercritical values of the Reynolds number. The question of whether these oscillations may be classed as "resonant" or "self-induced" is to some extent a matter of definition. The writer believes, however, that the aerodynamic action is better treated as a force proportional to, but out-of-phase with, the displacement of the motion (that is, as a damping force) rather than as an external independent simple harmonic forcing function of frequency prescribed by the Strouhal number. Evidence in support of this view is provided by the results of some measurement of the aerodynamic excitation on a circular cylinder carried out at the National Physical Laboratory. For the range of reduced velocity V/ND between 4.5 and 10 approximately, the aerodynamic damping was negative while for values of V/ND outside this range the damping was positive. Positive damping, of course, indicates that the body was giving up its energy of oscillation to the windstream; on the other hand, the forced oscillation theory would predict that the windstream would supply some energy to the body for all values of V/ND. The aerodynamic excitation (i.e. negative damping) was found to be dependent on the amplitude of the motion; it decreased with amplitudes so that the oscillations were of limited amplitude. The aerodynamic damping coefficient can be related to the concept of a Karman coefficient C_K if it is assumed that the maximum excitation corresponds to a resonance. The energy inputs per cycle derived from two concepts may then be equated. The results mentioned above indicate that C_K for "resonant" conditions is also dependent on amplitude; the value increasing initially with increase in amplitude and then decreasing fairly rapidly. At the maximum value, found with an amplitude of 0.15 diameter and V/ND = 4.75, the NPL results yield a value of $C_K = 0.7$ approximately. The value of Reynolds number for these tests was about 20 000.

a. Proc. Paper 1709, July, 1958, by Glenn B. Woodruff and John J. Kozak.

1. Aerodynamics Div., National Physical Lab., Middlesex, England.

HENRI PERRIN,[1] A. M. ASCE.—The drag coefficients for elementary shapes are very useful data and the values shown on Fig. 2 should be supplemented by some other informations, such as:

(a) Ratio of flange width to web length for WF- and I-beams
(b) Ratio of leg lengths for angle with unequal legs
(c) Reduction coefficients for beams of finite length.

The values given for shape No. 3 correspond, according to the Belgian(1) and Swiss(2) Coes, to a WF-beam with flange width equal to web length. On the other hand, the shape No. 4 will correspond to an I-beam with flange width approximately half of the web length.

Test data should also be interpreted with some care. The Swiss Code recommends to use for the coefficient C_L, shape No. 4 (I-beam) a value of ± 0.50 since a very slight yaw will at once produce an uplift not considered under the ideal conditions of a test. The writer would also like to point out the discrepancy between the two values given in Fig. 1a and 1b for a plate with $\lambda = 1.0$. The variations of the Reynolds number have been fairly well established for round and sharp-edged members, and the coefficient 6,380 vh corresponds to an athmospheric pressure of 30" Hg and a temperature of 60° F. Since the shape factor for round members may drop by as much as 50% with increasing Reynolds number, it should be noted that the Reynolds number varies between 8,200 vh for 31" Hg and 0° F. and 5,350 vh for 29" Hg and 100° F.(3)

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1. "L'action du vent sur les constructions" by L. Blanjean, "Ossature Metallique", Bruxelles, February 1949.
2. "Normen für die Belastungsannahmen, die Statischen Berechnungen, die Abnahme, die Ueberwachung und den Unterhalt der Bauten", Schweiz. Ing. und Arch. Verein, Entwurf 1951, No. 160.
3. "Technical Aerodynamics" by Wood, McGraw Hill, 1935.

1. Sr. Designer, Barstow, Mulligan & Vollmer, New York, N. Y.

WIND FORCES ON STRUCTURES: FORCES ON ENCLOSED STRUCTURES[a]

Discussions by C. Scruton and Henri Perrin

C. SCRUTON.[1]—Following a recent incident when the wind removed the roofs of some houses, there has been some interest at the NPL in the high-suctions which occur locally on the roofs of flat-roofed buildings in winds directed cornerwise. Suctions as high as $C_p = -6.0$ were found during model tests by N. Chien et al(1) and by C. Salter.(2) The latter identifies the high suctions with the conical vortices which start at the corner and suggests that they may be avoided by the use of simple spoilers which prevent the development of well-defined vortices. The Swiss Building Code, while perhaps the best and most detailed of the national codes, does not indicate the full extent of those very high local suctions. It is, however, probably still adequate for most design purposes.

REFERENCES

1. N. Chien et al. Wind Tunnel Studies of Pressure Distribution on Elementary Building Forms. Iowa Institute of Hydraulic Research 1951.
2. C. Salter. Wind Loading on Flat-roofed Buildings. Engineering, Oct. 17, 1958.

HENRI PERRIN,[2] A. M. ASCE.—The old Duchemin rule for inclined roofs has been much improved as shown on Fig. 1. For the sake of comparison, here are the values proposed by the French Code(1) for a closed structure:

Leeward wall and roof slope:	$C_{pa} = -0.5$
Windward roof slope:	$C_{pa} = 0.03\alpha - 1.0$
with $-0.7 < C_{pa} < +0.8$	
Windward wall	$C_{pa} = +0.8$

As such, these coefficients are very close to those proposed by the ASCE Committee. The French Code however adds to the above values a uniform suction, or pressure, acting inside the building

$$C_{pa} = \pm 0.3$$

a. Proc. Paper 1710, July, 1958, by Thomas W. Singell.
1. Aerodynamics Div., National Physical Lab., Middlesex, England.
2. Sr. Designer, Barstow, Mulligan & Vollmer, New York, N. Y.

The total wind force applied on the whole structure remains the same.

The translation of part of the Swiss Code will certainly provide useful informations for the designer. It seems however worthwhile to mention:

- drag and lift coefficients for wire and cable under any angle of attack(2)
- shape factors for cylindrical reservoir with conical roof (3)

Although the data available to the designer are very detailed they are sometimes not sufficient and tests in a wind-tunnel are the only means to achieve a certain degree of accuracy in determining the wind forces. Two of the latest investigations are also worth mentioning.(4,5)

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1. "Régles définissant les effets de la neige et du vent sur les constructions", Ministére de la Reconstruction et de l'Urbanisme, Paris, 1947.
2. "Engineering Aerodynamics", by W. S. Diehl, Ronald Press Co., 1930.
3. "L'action du vent sur les constructions", by L. Blanjean, Ossature Métallique, Bruxelles, Feb. 1949.
4. "Aerodynamic study of a cooling tower in the form of a hyperboloid of revolution" by G. Golubovic, Intern. Ass. for Bridge and Str. Eng., Zürich, 1957.
5. "Aerodynamic study of the French Pavillon at the Brussels Fair" by P. Vallée, R. Pris, C. Bonvalet, Intern. Ass. for Bridge and Str. Eng., Zürich, 1958.

WIND FORCES ON STRUCTURES: PLATE GIRDERS AND TRUSSES[a]

Discussions by C. Scruton and Henri Perrin

C. SCRUTON.[1]—The simple formulae introduced by the author for computing the drag coefficients of trusses offer considerable advantages, and the writer would like to make a few comments on checks on the formulae which are available from ad hoc work carried out at the NPL. For single trusses the value $C_D = 1.8$ appears to be rather conservative; a value of 1.7 or 1.6 is preferred and indeed this value corresponds more closely to those given by the author in Fig. 7, and with the values indicated by Cohen and Perrin[(1)] in Fig. 6 of their ASCE paper 1355. For the square tower with b/d = 1 the NPL data confirms the authors simple formula $C_D^2 = 3\sqrt{2}\phi$ provided the space between the trusses is "empty". It also fits the curve of experimental points given in Fig. 12 of Cohen and Perrin's paper.[(1)] However, this relation only holds for winds normal to a face and further relations are required to allow for vertically inclined and quartering winds. Also masts, especially those for radio purposes, are usually required to carry a variety of internal fittings such as cables, ladders and lift shafts, etc., and the increase in the drag is substantial and difficult to assess. With internal fittings the simple formula quoted above tends to underestimate the drag.

A fairly comprehensive series of measurements of the wind forces on solid deck plate girder bridges was carried out in 1951 by Williams, Nixon and Skelton.[(2)] The results were not published but they were used in the formulation of the British Standard on Plate Girder Bridges. The tests corresponded to an infinite aspect ratio and showed the following marked "Eiffel effect".

b/d	0	1	2	4	6
C_D/C_{D_o}	1	1.06	0.79	0.74	0.72

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1. E. Cohen and H. Perrin, Design of Multi-level Guyed Towers: Wind Loading. ASCE Paper 1355, Vol. 83, No. ST5, September 1957.
2. R. C. Baird, "Wind-Induced Vibrations of a Pipe-line Suspension Bridge and its Cure," Trans. ASME, Vol. 77, No. 6, August, 1955.

a. Proc. Paper 1711, July, 1958, by W. Watters Pagon.
1. Aerodynamics Div., National Physical Lab., Middlesex, England.

HENRI PERRIN,[1] A. M. ASCE.—The equation proposed by the author for the drag coefficient of a single truss is somewhat different of that proposed by Buchegger, namely:

$$C_D' = 1.6\phi$$

with $0.2 < \phi < 0.9$ and confirmed by the numerous tests of Flachsbart and others. It should also be noted that most of the values shown on Fig. 7 converge toward this value for $\phi = 0$.

On the other hand, the following equations were proposed(1) for the drag coefficient of towers:

square tower : $C_D' = 4\,\phi - 5\,\phi^2$

triangular tower: $C_D' = 3.65\,\phi - 4.65\,\phi^2$

both for $0.1 < \phi < 0.4$, which is the usual range for such structures. The available test data confirm fairly well the above equations. There seems to be no reason to have two different drag coefficients, 1.55 and 1.8 as shown on Fig. 6, for single trusses used either in a tower or in a different structure.

It is generally agreed to use for round member trusses, and towers, 2/3 of the value for flat member trusses and towers. The writer, however, obtained for $\phi = 0.22$, ratios ranging from 0.53 to 0.70 from the Guggenheim tests. For $\phi = 0.14$, the ratio was 0.61.

Data concerning the shielding coefficient and the variations of drag with yaw (or pitch) may be found in some of the references listed in the paper previously mentioned.

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1. "Design of multi-level guyed towers: Wind loading", by E. Cohen and H. Perrin, Proc. ASCE, Paper No. 1355, Sept. 1957.

1. Sr. Designer, Barstow, Mulligan & Vollmer, New York, N. Y.

WIND FORCES ON STRUCTURES: STRUCTURES SUBJECT TO OSCILLATION[a]

Discussion by C. Scruton

C. SCRUTON.[1]—Although we have a workable (but perhaps limited) understanding of vortex-excited oscillations of bluff bodies in the sub-critical range of Reynolds number, no basic explanation of the excitation at super-critical values, which apparently produces oscillations at reduced frequencies differing from the Strouhal frequency for low values of Reynolds number and which conflict with the usually assumed aperiodic character of vortex shedding in super-critical flow, has yet been satisfactorily substantiated. Unfortunately few wind-tunnels are suitable to investigate this problem because of the limited value of Reynolds number attainable. Possible causes for the oscillations of multiple stacks might be the buffeting due to the eddying wake of a windward stack, or the mutual interference of the airflow between adjacent stacks when oscillating in winds more normal to the plane of the stacks. This latter type of oscillation is known to occur with boiler heat exchanger tubes.

Factors influencing the aerodynamic stability of structures when oscillating in an uncoupled mode are simply

(i) The aerodynamic shape,
(ii) The natural frequency, and
(iii) The structural damping.

Preventive measures based on (ii) and (iii) have been used for tall stacks but their application is not always easy or convenient. In modern suspension bridge design aerodynamic stability is achieved by attention to the aerodynamic shape. Modifications to the aerodynamic shape for preventing the wind excited oscillations of cylindrical structures such as tall stacks, pipe lines, hangers, etc., have been suggested by Baird,(1) by Price,(2) and by Scruton and Walshe.(3) A brief description of the last named method might be of interest since a description is not available in a journal or in published transactions. It has been found experimentally that the aerodynamic excitation due to vortex-shedding on circular cylinders can be reduced and finally eliminated for all wind speeds by an arrangement of strakes wound in helix formation on to the surface of the cylinder. The benefit to be derived from this arrangement depends on the number of strakes fitted and on their height and pitch. One effective arrangement consisted of a three start strake of height about 1/8 diameter and of pitch 15 diameters.

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a. Proc. Paper 1712, July, 1958, by F. W. Farquharson.
1. Aerodynamics Div., National Physical Lab., Middlesex, England.

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1. R. C. Baird, Wind-induced Vibrations of a Pipe-line Suspension Bridge and its Cure. Trans. ASME Vol. 77, No. 6, August 1955.

2. P. Price. Suppression of the Fluid-induced Vibration of Circular Cylinders. Proc. ASCE 1030, Vol. 82, July 1956.

3. C. Scruton and D. E. J. Walshe. A Means for Avoiding the Wind-excited Oscillations of Structures with Circular or Nearly Circular Cross-section. Report NPL/Aero/335 (unpublished).

ANALYSIS OF FINITE BEAMS ON ELASTIC FOUNDATION[a]

Corrections to Discussion by Alexander Dodge

CORRECTIONS TO DISCUSSION BY ALEXANDER DODGE,[1] M. ASCE.—In the January, 1959, Journal of the Structural Division, on p. 160, in the sixth line of text following Eq. (6), the minus sign after xb should be changed to an equals sign. On p. 168, in the line just above Table 2, the P_b-term in the expression for y should be raised to the third power. On p. 172, in the first line of text following the six lines of retyped text, the reference to Fig. 5 should be changed to Fig. 6.

a. Proc. Paper 1722, July, 1958, by D. C. Gazis.
1. Tech. Asst., Corp. of Engrs., Portland, Ore.

BIAXIALLY LOADED REINFORCED CONCRETE COLUMNS

Discussion by Tung Au

TUNG AU,[1] A. M. ASCE.—The paper presents a thorough analysis of stress and strain distribution in the inelastic range of biaxially loaded reinforced concrete columns. The significance of this basic approach lies not only in the possibility of its application to column sections of various shapes but also in its clarification of basic criteria of failure of such members.

The authors correctly stated the limitations of the method proposed earlier by the writer. Of these limitations, the assumption for locating the point of application of resultant tension of the steel area is most objectionable in that it may introduce appreciable error if judgment is not exercised in locating the reinforcing bars. To a lesser extent, such error also exists in most columns eccentrically loaded in one direction encountered in actual practice. For a rectangular column section, the reinforcing bars are frequently placed along all four sides of the section if bending stress is relatively small in comparison with tensile stress in steel; or the bars may be arranged in more than one layer on the two sides parallel to the axis of bending if bending stress becomes predominant. In either case, the point of application of resultant tension is not identical with the centroid of tensile reinforcing bars. Nor will yielding begin in all tensile bars at the same time. For a square or circular column with a round core, similar situation prevails. Hence, the ultimate strength design formulas for combined bending and axial load in the ACI Building Code[2] involve certain degree of approximation in this regard. The present paper provides an excellent means of checking the accuracy of these formulas as well as those for unsymmetrical bending proposed by the writer.

The method suggested in the paper is also useful in the study of inelastic buckling of centrally or eccentrically loaded long concrete columns. The versatility of this method will more than compensate its lengthy trial and error procedure which can eventually be overcome by the use of digital electronic computer.

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a. Proc. Paper 1865, December, 1958, by Kuang Han Chu and Algis Pabarcius.

1. Associate Prof. of Civ. Eng., Carnegie Inst. of Technology, Pittsburgh, Pa.
2. "Building Code Requirements of Reinforced Concrete (ACI 318-56)", American Concrete Institute, Sections A609 and A610.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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c. Discussion of several papers, grouped by divisions.

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NEWS OF THE STRUCTURAL DIVISION OF ASCE

JOURNAL OF THE STRUCTURAL DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES
STRUCTURAL DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

March, 1959

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ASCE CONVENTION, CLEVELAND, MAY 3-7, 1959

The Structural Division will sponsor five half-day technical sessions at the Society's Summer Convention, to be held at the Cleveland Hotel, May 3-7, 1959.

Carl L. Erb, Structural Division Local Chairman on Sessions Program, reports that the program is essentially complete. Herbert Rothman has prepared the program on Electronic Computation for the two sessions on Tuesday. The remaining three sessions are on Nuclear Structures and Materials. Wednesday's program has been prepared by H. M. Glen and Thursday's by John Goldberg. These are the first sessions at a national meeting of the Society to be devoted mainly to Nuclear Structures and Materials. These men have prepared a very fine program worthy of the support and attendance of the Division members. An outline of the tentative program follows:

Tuesday Morning—May 5, 1959 (Sponsored Jointly with Engineering Mechanics Division)

ELECTRONIC COMPUTATION

1. Solutions of Large Systems of Linear Equations.

Note: No. 1959-11 is part of the copyrighted Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Vol. 85, ST 3, March, 1959.
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A. H. Morris, Department of Electrical Engineering, University of Houston, Houston, Texas.

2. Analysis of Conical Shells by Electronic Computer.
   John Goldberg, M. ASCE, Department of Civil Engineering, Purdue University, Lafayette, Indiana.
3. Dynamic Response of Multistory Elasto-Plastic Frame by Computer.
   Glen Berg, A. M. ASCE, Engineering Research Institute, University of Michigan, Ann Arbor, Michigan.
4. Joint Displacements in Ideal Trusses.
   Donald DaDeppo

Tuesday Afternoon—May 5, 1959

ELECTRONIC COMPUTATION IN BUILDING DESIGN

1. Computer Design of a Multistory Frame Building.
   A. M. Lount, A. M. ASCE, Consulting Engineer, A. M. Lount & Assoc., Toronto, Ontario.
2. Computer Design of Power Plant Framing.
   John H. Wells, M. ASCE, Jackson & Moreland, Inc. Boston, Massachusetts.
3. Analysis of Rigid Frames.
   Ardis M. White, A. M. ASCE, Department of Civil Engineering, University of Houston, Houston, Texas.
4. Design of Prestressed Beams by Computer.
   Joseph Bonasia, J. M. ASCE, Ozone Park, New York

Wednesday Afternoon—May 6, 1959

The subject under consideration in the Hot Laboratory Subsection Session is "Divergent Philosophies Affecting the Layout and Structural Features of Alpha-Gamma Hot Cell Design."

H. M. Glen as Chairman of the Session will open the program with an introductory discussion entitled "General Aspects of Plutonium Confinement Within Hot Cells."

The list of papers by title and author are as follows:

1) "Plutonium Confinement Within a Typical Beta-Gamma Hot Cell." by George E. Parker, Project Engineer, Vitro Engineering Div., 225 4th Ave., New York 3, New York.
2) "Alpha-Gamma Hot Cells Utilizing Maximum Personnel Set Up and Maintenance." by John M. Ruddy, Chief Engineer, Architectural Planning Div., Brookhaven National Laboratory, Upton, Long Island, New York.
3) "Alpha-Gamma Hot Cells Utilizing Maximum Remote Set-Up and Maintenance." by K. R. Ferguson, Remote Control Engineering Div., Argonne National Laboratory, Lemont, Illinois.
4) A question and answer period will follow the third paper.

All of the above personnel are members of the Hot Laboratory Sub-Committee of the "Committee on Nuclear Structures and Materials," of the Structural Division. The handling of alpha emitters in a dry state is a problem of ever increasing importance to Nuclear installations. And, the best manner of approach to this problem is still a matter of serious dispute. There is no meeting of minds, as yet, on the safest or most efficient facility to design

for handling fuel elements, in pile loops or other fission products in a dry state. Whether to attempt to utilize dry boxes within existing type beta-gamma hot cells, whether to so design the alpha-gamma cell bank that personnel protected by frog-man type clothing, can be used for inner cell equipment set up or maintenance, or, whether to so design the alpha gamma cell bank that personnel are never used for inner cell set up or maintenance—that is the question. In this Session the good and bad points of all three solutions will be discussed.

Thursday Morning—May 7, 1959 (Sponsored jointly with Construction Division)

The sessions will open with a paper, "Containment Problems," by Dr. Stuart McLain, formerly Associate Director of Argonne National Laboratory and now a Nuclear Consultant, in which he will discuss the history of the containment problem, reasons for containment, types of vessels and applicability of existing codes. He will be followed by John A. Bailey of the Atomic Power Equipment Department, General Electric Company, who will discuss "Pressure-Volume Considerations for Reactor Containment Vessels;" Francis B. Porzel of the Armour Research Foundation, who will present "A New Approach to Heat and Power Generation from Contained Nuclear Explosions;" and Reuben R. Alvy, M. ASCE, Chief, Engineer, Nuclear Division, Holmes and Narver, who will discuss "Seismic Aspects of Nuclear Facilities."

Thursday Afternoon—May 7, 1959

At the afternoon session, Norman R. Zabel of Stanford Research Institute will present information on "Containment of Fragments from Runaway Reactors;" and John F. Stolz, A. M. ASCE and his associates of Atomics International, will discuss "Containment Criteria for Organic Moderated Power Reactors." They will be followed by R. F. Devine of the Bettis Atomic Power Division, Westinghouse Electric Corporation and John J. Niland, A. M. ASCE of Stone and Webster Engineering Corporation, who will present two papers on the containment sphere for the Shippingport, Pennsylvania, Pressurized Water Reactor. Mr. Devine will discuss the basis for design of this containment sphere, and Mr. Niland will discuss the execution of the design.

COMMITTEE ON ELECTRONIC COMPUTATION

Papers from the Conference on Electronic Computation

In response to the unprecedented demand expressed by the Conference participants, the twenty five papers presented at the Conference on Electronic Computation held in Kansas City, Missouri on November 20-21, 1958 are being offered in a single paper-bound volume. This special edition is composed of the twenty two technical papers and the welcome address, the keynote address, and the luncheon address, all as delivered at the conference. The prices (post-paid) are as follows:

	Members	Non-Members
First Copy	$ 5.00	$10.00
Additional Copies	$10.00	$10.00

If you wish to order a copy of this publication please use the order form at the end of the Newsletter.

New Task Committees

The two new Task Committees met in Kansas City to formulate statements of purpose and to outline their work. The statements of purpose are given below.

1. Task Committee on Educational Aspects of Electronic Computation (Elwyn King, Chairman)

   Purpose: (a) To review the background and training necessary for the efficient use of digital computation by civil engineers.
   (b) To report on courses in programming, coding and numerical analysis now being given or scheduled by manufacturers, management consulting firms, and universities for informational use by Society members.
   (c) To encourage establishment of short training courses in the above subjects by universities or other groups and to report on methods of implementing this aim.
   (d) To study the impact of electronic computation on engineering education and submit findings or recommendations on this subject to the Committee on Electronic Computation for possible referral to the Committee on Engineering Education.

2. Task Committee on Program Interchange (James Michalos, Chairman)

   Purpose: (a) To determine the status of the interchange of information relating to computer programs among members of the profession and to formulate a set of recommendations for a policy on the exchange of programs.
   (b) To determine the effect of computer use on engineering practice, and, if necessary, prepare recommendations to be submitted to the Committee on Conditions of Practice.

NEW $5,000 RESEARCH FELLOWSHIP AVAILABLE

A new A.S.C.E. Research Fellowship has been established by the Board of Direction, for the purpose of aiding in the creation of new knowledge for the benefit and advancement of the science and profession of Civil Engineering. The grant, in the amount of $5,000, is to be made annually from current income of the Society.

Preference will be given to research that is basic in nature and concept, rather than applied, developmental, or designed to extend or eleborate information. Research requiring an extensive testing program will not be considered.

Rules to guide applicants can be found in the 1959 ASCE Official Register. Inquiries regarding the fellowship should be addressed to William H. Wisely, Executive Secretary, American Society of Civil Engineers, 33 West 39th Street, New York 18, New York.

MIDWESTERN CONFERENCE ON FLUID MECHANICS AND SOLID MECHANICS

A number of technical societies and research organizations have joined with the University of Texas in co-sponsoring the Midwestern Conference on Fluid Mechanics and Solid Mechanics to be held on the Austin campus on September 9-11, 1959. This meeting, which will be the Sixth Conference on Fluid Mechanics and the Fourth Conference on Solid Mechanics, forms another element in the series held in the midwestern region of the United States. Previous conferences, at approximately two-year intervals, have been held at the Universities of Illinois, Ohio State, Minnesota, Purdue, and Michigan.

Although the Conferences are held on a regional basis, there are no restrictions on attendance or on the residence of authors of technical papers. A General Conference Committee for handling initial arrangements consists of Dr. Walter L. Moore, Professor of Civil Engineering; Dr. Enrico G. Volterra, Professor of Engineering Mechanics; and Dr. Milton J. Thompson, Professor of Aeronautical Engineering. Abstracts and titles of proposed papers should be submitted to Professor Thompson as chairman before March 1, 1959. Complete manuscripts will be required by May 1, 1959, so as to make possible the distribution of the Proceedings at the opening of the Conference. Inquiries concerning other details of the Conference should also be addressed to Professor Thompson.

The University of Texas, as a part of its support of the Conference, has arranged for a series of invited lectures on important aspects of mechanics. Dr. Samuel A. Schaaf of the University of California will present a lecture on "Recent Progress in Rarefied Gas Dynamics Research," while Dr. Lloyd H. Donnell of the Illinois Institute of Technology will talk on "New Developments in Shell Theory." At a dinner meeting, Dr. Sydney Goldstein of Harvard University will speak on "Recent Progress in Applied Mechanics."

Arrangements are being made for entertainment programs for conferees and their ladies. Tours of University laboratories and a visit to the Southwest Research Institute in San Antonio are also being planned.

FOURTH U. S. NATIONAL CONGRESS OF APPLIED MECHANICS

The Fourth U. S. National Congress of Applied Mechanics will be held on the Berkeley campus of the University of California during June 18-21, 1962. Research workers in the theoretical and applied mechanics of solids and fluids are invited to submit papers for consideration by the Editorial Committee. Inquiries regarding the Congress should be addressed to Professor W. Goldsmith, Secretary, Division of Mechanics and Design, University of California, Berkeley 4, California.

STRUCTURAL DIVISION JOURNAL

News items for the May issue of the Newsletter should be submitted to the Editor before the end of March. Comments on and suggestions for the improvement of the Newsletter will be welcomed.

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